GEOTECHNICAL INVESTIGATION REPORT

MAIN AVENUE AND MADRONE PIPELINES RESTORATION PROJECT
SANTA CLARA COUNTY, CALIFORNIA

For

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1.0 INTRODUCTION

This report presents the results of our geotechnical engineering investigation performed for the proposed “Main Avenue and Madrone Pipelines Restoration Project” in Santa Clara County, California, hereinafter referred to as “PROJECT”. The work was performed in general accordance with the scope of work outlined in our proposal to Santa Clara Valley Water District (District).

This report presents the results of our geotechnical investigation and provides recommendation concerning the geotechnical engineering aspects of the project. The approximate location of the project site is shown on the Vicinity Map (Plate No. 1) and Project Location Map (Plate No. 2).

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for direct bidding purposes or for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be the sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2.0 PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general subsurface soil conditions at the project site, to evaluate their engineering properties, and to present recommendations for the geotechnical design of the proposed project for the restoration of the existing pipeline.

The scope of work performed for this investigation has included:

- Reviewed of readily available geologic and soil literature in the vicinity of the site including review of any as-built data.
- Performed a site reconnaissance to review the site conditions
- Performed a field exploration program. A total of 14 exploratory (Borings B-01 through B-14) borings were explored with their locations approximately evenly distributed along the alignment of the proposed pipeline. The proposed locations of the borings have been reviewed and finalized by the District. The depth of the drilled boring is approximately

21.5 feet below existing ground surface. Obtained representative samples and logging soil materials encountered in the exploratory borings.

- Performed laboratory testing on selected soil samples retrieved from the field exploration. Laboratory tests include

  a) Moisture content; (California Test Method 226);
  b) Density (Based on mass/volume relationships);
  c) Plastic Limit, Liquid Limit & Plastic Index (California Test Method 204);
  d) Grain Size Distribution Analysis (ASTM Test Method D422-63);
  e) Unconsolidated Undrained (UU) Triaxial Test (ASTM D 2850);
  f) Corrosion - 7 tests equally spaced along the alignment and at approximate pipe invert depth – (Sulfate content, chloride content, resistivity and pH) (California Test Methods 417, 422, and 643);
  g) R-Value (ASTM Test Method D 2844);
  h) Modified Proctor Compaction (ASTM Test Method D 1557).

- Evaluated geologic and seismic hazards conditions of the project site

- Performed engineering analysis with the field and laboratory data and preparation of this Geotechnical Investigation Report (Report) based on the ground investigation information from the exploratory soil borings. Developed recommendations for geotechnical design, including:

  a) Trench backfill.
  b) Design groundwater elevation.
  c) Evaluation of liquefaction potential.
  d) Modulus of Soil Reaction, E' (for native and composite subgrade consisting of compacted trench backfill/native.
  e) Pipe friction and thrust block design.
  f) Structural pavement section with provided Traffic Index.
  g) Buoyancy effect on pipes
  h) Construction consideration.
  i) Other design issues which may be relevant for this project.
  j) Guidelines for protection of adjacent utilities.

k) Discussions of the "Trench Widths".

3.0 PROJECT BACKGROUND

The District proposes to replace the existing pipelines with approximately 13,960 feet of steel pipe, between 30 and 36 inches in diameter, at the following locations in the City of Morgan Hill.

Main Avenue Pipeline

1. Reach 1 – Along Cochrane Road between near Coyote Creek to Cochrane Road/Half Road intersection.
2. Reach 2 – Along Cochrane Road, E. Main Avenue and access road to Main Avenue Ponds between Cochrane Road/Half Road intersection and Main Avenue Ponds.

Madrone Pipeline

3. Reach 3 – Along Half Road between Cochrane Road/Half Road intersection and Madrone Channel.

The new pipe invert will be between 5 feet and 9.5 feet below existing grade and will be installed by open-cut construction method. The trench will be backfilled with the soil consistent with the District's backfill requirements.

Our recommendations presented in this report are based on the above information. Any major deviation should be reported to PARIKH for further consideration.

4.0 PREVIOUS STUDIES

The following geologic and geotechnical engineering reports were available from the District for review:

a) Anderson Dam Seismic Stability Evaluation Report (SSE-1A), June 2011, by AMEC Geomatrix.

b) Technical Memorandum No. 6 Phase 1 Fault Rupture Hazard Evaluation - Seismic Stability Evaluation of Anderson Dam Agreement No. A3087A, File 4073, September 2009, by AMEC Geomatrix.

5.0 SITE CONDITIONS

Based on the “Map and General Plan for Main Avenue and Madrone Pipelines Restoration Project 30% Design Submittal” (30% Submittal) provided by the District, the topography in the vicinity of the project site is generally gently sloping from the southwest to the northeast and from the south to the north. The ground surface elevations of the borings vary from the minimum elevation of 383.5 at Boring B-01 (Condit Road/Half Road intersection) to the maximum elevation of 473.5 at Boring B-08 (Cochrane Road).

The restored pipelines will be along mainly Cochrane Road, Half Road and Main Avenue. According to City of Morgan Hill, Half Road is considered as a collector and Main Avenue is considered as an arterial. The area in the vicinity of the proposed restored pipelines is mainly residential and farmland.

6.0 FIELD EXPLORATION AND LABORATORY TESTING

The field exploration program was developed based on discussions with the District. A total of fourteen borings (Borings B-01 through B-14) were drilled to the depth of 21.5 feet.

The approximate locations of the borings are shown in the “Boring Location Map”, Plate No. 3. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration are shown on the "Log of Test Borings".

Laboratory tests were performed on selected soil samples collected during field exploration to evaluate the physical and engineering properties of the subsurface soils. The laboratory test methods and test results are presented on plates included in Appendix B. Laboratory test results for moisture content, total unit weight, unconfined compression, Plasticity Index and grain size classification of the soil samples are presented on the Log of Test Borings in Appendix A.

It should be noted that the descriptions of the soils encountered and relevant boring information presented on the Log of Test Borings depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the Log of Borings. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the boring locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter unforeseen variations in the subsurface soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

7.0 SUBSURFACE SOIL CONDITIONS

Based on the boring data of Borings B-01 through B-14, the subsurface soil conditions encountered in the soil borings (depth of 21.5 feet) within the project area are summarized in the table below.

TABLE 1 - SUMMARY OF SUBSURFACE SOIL CONDITIONS

Boring No.	Approximate Elev. (ft)	Subsurface Soil Conditions
B-01	383.5	Approximately 11 feet of very stiff to hard lean clay with gravel, underlain by medium dense to dense clayey sand with gravel to the boring depth.
B-02	388.5	Approximately 9 feet of medium stiff to very stiff lean clay with gravel, underlain by approximately 5 feet medium dense clayey sand with gravel, underlain by very stiff lean clay to the boring depth.
B-03	393.5	Approximately 9 feet of hard lean clay, underlain by very stiff to hard lean clay with gravel to the boring depth.
B-04	398.0	Approximately 4 feet of hard silt, underlain by approximately 5 feet of hard lean clay, underlain by approximately 5 feet of dense clayey gravel, underlain by hard lean clay with gravel to the boring depth.
B-05	408.0	Very stiff to hard lean clay with sand to the boring depth.
B-06	415.0	Approximately 5 feet of very stiff lean clay with sand, underlain by approximately 3 feet of medium dense clayey sand with gravel, underlain by hard lean clay to the boring depth.
B-07	423.0	Approximately 8 feet of medium dense to very dense silty sand with gravel, underlain by approximately 10 feet of dense poorly-graded sand/silty sand, underlain by medium dense silty gravel with sand to the boring depth.
B-08	473.5	Approximately 13 feet of medium stiff to very stiff lean clay with sand, underlain by approximately 4 feet of dense clayey sand with gravel, underlain by hard sandy silt to the boring depth.
B-09	450.0	Approximately 9 feet of very stiff to hard lean clay with gravel, underlain by hard claystone to the boring depth.
B-10	424.5	Approximately 13 feet medium stiff to very stiff lean clay, underlain by dense to very dense silty sand with gravel to the boring depth.
B-11	414.0	Approximately 14 feet medium stiff to hard lean clay with sand, underlain by hard lean clay with gravel to the boring depth.
B-12	397.5	Approximately 4 feet of stiff lean clay with sand, underlain by medium dense to dense silty sand, underlain by approximately 4 feet of dense clayey sand, underlain by hard lean clay with gravel to the boring depth.
B-13	394.5	Approximately 4 feet of hard lean clay with sand, underlain by approximately 5 feet of hard gravelly lean clay, underlain by approximately 9 feet of hard lean clay with sand, underlain by hard gravelly lean clay to the boring depth.
B-14	395.0	Approximately 8 feet of very stiff to hard lean clay with sand, underlain by approximately 10 feet of dense to very dense silty sand with gravel, underlain by dense clayey gravel with sand to the boring depth.

Groundwater was not encountered in any of the fourteen borings during drilling in August 2015 with the maximum boring depth of 21.5 feet. Also, none of the soil samples collected from these borings was described as “wet”. Monitoring wells were installed in Borings B-01, B-07 and B-14 in August 2015 to allow groundwater monitoring. Groundwater was measured in the three monitoring wells in October and December 2015 with the following summary:

TABLE 2 - SUMMARY OF GROUNDWATER MONITORING

Boring Number	Encountered Groundwater Level		
	Date Measured	Depth (ft)	Approx. Elev. (ft)
B-01	10/15/2015	18.9	364.6
	12/24/2015	19.0	364.5
	03/10/2016	18.9	364.6
B-07	10/15/2015	20.3	402.7
	12/24/2015	16.3	406.7
	03/10/2016	13.1	409.9
B-14	10/15/2015	12.0	383.0
	12/24/2015	13.6	381.4
	03/10/2016	14.5	380.5

Based on the “Seismic Hazard Zone Report for the Morgan Hill and Mt. Sizer 7.5 Minute Quadrangles, Santa Clara County, California”, the depth to groundwater at the project site is estimated to be in the range of 50 feet to 80 feet. The “Historically High Ground Water” (Plate No. 4) presents the contours of depth to groundwater in the vicinity of the project site. According to the “Santa Clara Valley Water District 10-Year Groundwater Elevation Data from 1/2006 to 1/2016”, groundwater was measured to the depth of 15.5 feet (NGVD29 Elev. 374.6 feet) on December 22, 2015 from the monitoring well installed at E. Main Avenue.

The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, variations in yearly rainfall, water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other environmental factors that may not be present at the time of the investigation. Based on the measured groundwater levels, a groundwater depth of 10 feet was assumed for engineering design purposes only.

Note that the explorations were performed during the worst drought period that California has experienced. Ground water elevation could significantly vary in the event of a ‘normal’ rainfall period or following an El Nino period. Also, since groundwater may take time to recharge or react to such changes, the potential fluctuations due the extreme conditions as noted above may or may not be observed during immediately after the commencement of or during construction.

Therefore, for the proposed project, it is imperative that the contractor is aware of such potential change in groundwater level and should not solely rely on such transient measurements from the boring/CPT data. It may be prudent to make conservative assumptions in the construction program to account for the potential variation and/or to verify the groundwater level prior to the construction."

Groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, fluctuation of water elevations in the nearby creeks, surface and subsurface flows, ground surface run-off, and other environmental factors, which may not be present at the time of the investigation. Also the severe drought period (during the investigation) followed by the El Nino winter can significantly impact the water elevations. This should be considered in the design and construction of the proposed project.

8.0 GEOLOGY

The project site is in the southern portion of San Francisco Bay area in the Coast Range geomorphic province of California. The Coast Range forms a nearly continuous topographic barrier between the California coastline and the San Joaquin Valley. The Coast Range in this region is a double chain of mountain running north-northwest. Three prominent geologic blocks dominate the San Francisco Bay Area: the Santa Cruz Mountains (western block), the San Francisco Bay (central block), and the East Bay Hills/Diablo range (eastern block).

General geologic features pertaining to the project site and its vicinity were evaluated with reference to the geologic map titled "Geological Maps of the Mount Sizer and Morgan Hill Quadrangle, Santa Clara County, California" 2005, Dibblee, T.W., and Minch, J.A.; Dibblee Geological Foundation, Maps DF-15980-165, Scale: 1:24000. Based on the map, the project site is underlain by alluvial gravel, sand and clay of valleys (Qa) (Holocene). The general geology of the project area is shown on the Geologic Map, Plate No. 5. We also referred to "Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Region, California" by Witter et al (Plate No. 9). Per this publication; the descriptions of the main quaternary geologic units are as follows:

Qhf- Holocene alluvial fan deposits. Sediment deposited by streams emanating from mountain canyons onto alluvial valley floors or alluvial plains, including debris flow, hyper-concentrated mudflow, and braided stream deposits. Alluvial fan sediment includes sand, gravel, silt, and clay, and is moderately to poorly sorted, and moderately to poorly-bedded.

Qf- Latest Pleistocene to Holocene alluvial fan deposits. This unit is mapped on gently sloping, fan-shaped, relatively undissected alluvial surfaces where the age of deposits is not known (either latest Pleistocene or Holocene in age) or where the deposits consist of thin patches of Holocene sediment overlying latest Pleistocene alluvial fan sediment. Fan sediment includes sand, gravel, silt, and clay, and moderately to poorly sorted, and moderately to poorly bedded.

Qpf- Latest Pleistocene alluvial fan deposits. This unit is mapped on alluvial fans where latest Pleistocene age is indicated by greater dissection than is present on Holocene fans, and/or the development of alfisols. Latest Pleistocene alluvial fan sediment was deposited by streams emanating from mountain canyons onto alluvial valley floors or alluvial plains and includes debris flow, hyper-concentrated mudflow, and braided stream deposits. Alluvial fan sediment typically includes sand, gravel, silt, and clay, and is moderately to poorly sorted, and moderately to poorly-bedded.

br- Early Quaternary and older (>1.4 Ma) deposits and bedrock, undifferentiated. Primarily Jurassic to Pliocene sedimentary, metamorphic, volcanic and plutonic rocks, and poorly consolidated Tertiary sediment.

9.0 EARTHQUAKE CONSIDERATIONS

9.1 Seismic Sources

The project site is located in a seismically active part of Northern California. Many faults, capable of producing earthquakes, exist in the San Francisco Bay Area. These faults may cause strong ground shaking at the project site.

Maximum moment magnitudes (Mmax) of some of the closest faults in the area are based on the Caltrans 2012 ARS Online Report. These maximum moment magnitudes represent the largest earthquake a fault is capable of generating and is related to the seismic moment. The fault data of the active faults in the project vicinity are summarized in the table below. A fault map is shown in Plate No. 6.

TABLE 3 - SUMMARY OF APPROXIMATE SITE-FAULT DISTANCES

Fault (Fault ID)	Maximum Moment Magnitude of Fault, M_{Max}	Fault Type	Site-to-Fault Distance, R_{rup}* (km)
Silver Creek (148)	6.9	Reverse	2.5
Calaveras (Central) 2011 CFM (151)	6.9	Strike Slip	3.7
Hayward (Southern extension)	6.7	Strike Slip	11.7
Cascade fault (153)	6.7	Reverse	12.9
Sargent Fault (northwestern section) (164)	7.0	Strike Slip	14.6
San Andreas (Santa Cruz Mts) 2011 (CFM) (158)	8.0	Strike Slip	18.5

*Closest distance to the fault rupture plane calculated by Caltrans ARS Online.

Based on the State of California “Special Studies Zones Morgan Hill & Mount Sizer Quadrant”, the project site is approximately over a mile from the “Alquist-Priolo Earthquake Fault Zone”. The “State Special Studies Zones Map (Fault)” (Plate No. 7) presents the location of the Alquist-Priolo Earthquake Fault Zone relative to the project site.

The Coyote Creek fault, with an estimated magnitude of 6.6, is in the vicinity of the project site. The Coyote Creek fault is an east-dipping reverse fault that offset the Santa Clara Formation. The Coyote Creek fault is considered to be “Conditionally Active” per State of California Division of Safety of Dams (DSOD) criteria.

9.2 Site Design Criteria

The design response spectrum was calculated per California Building Code (CBC) 2013 using "USGS Design Maps Website". CBC Acceleration Response Spectra is shown in Plate No. 8. The ground motion parameters and the Peak Ground Acceleration (PGA_M) are shown in the table below:

TABLE 4 - GROUND MOTIONS PARAMETERS PER CBC 2013

Location	Site Class	Rock Spectral Acc. (g)	Site Coefficients	Site-Modified Spectral Acc. (g)	Design Spectral Acc. (g)	PGA_M
Lat.: 37.1533 Long.: -121.62929	D	$S_S = 1.883$	$F_a = 1.0$	$S_{MS} = 1.883$	$S_{DS} = 1.255$	0.72
		$S_1 = 0.706$	$F_v = 1.5$	$S_{M1} = 1.059$	$S_{D1} = 0.706$	

9.3 Seismic Hazards

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction.

Seismic Ground Shaking

Based on available geological and seismic data, the possibility of the project site to experience strong ground shaking may be considered high. A peak ground acceleration of 0.72 g was calculated for the site in Section 9.2.

Surface Fault Rupture

As shown on the "State Special Studies Zones Map (Fault)", Plate No. 7, the "Alquist-Priolo Earthquake Fault Zone" is located more than a mile northeast from the project site. Since no active faults pass through the site, the potential for fault rupture is relatively low.

Liquefaction Susceptibility

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Saturated cohesionless sands and silts of low relative density are the type of soils that are usually susceptible to liquefaction. Clays are generally not susceptible to liquefaction. Gravels tend to drain well and are not usually susceptible to liquefaction.

The extent to which any of the subsurface soils underlying the proposed restored pipelines may be prone to liquefy is a function of their grain-size distribution, density, and level of saturation. The alluvial deposits beneath majority of the proposed restored pipelines appear to contain a mixture of fine-grained soil materials that would be less susceptible to liquefaction. However, pockets/lenses of loose to medium dense sand or non-plastic to low plasticity silt may exist at location for some of the proposed restored pipelines that would be subject to liquefaction or shaking-induced settlement.

Both the "Quaternary Deposits Map" (Plate No. 9) and "Liquefaction Susceptibility Map" (Plate No. 10) were referred to in the evaluation of the liquefaction susceptibility for the proposed restored pipelines at the project site. Both these maps are developed from Witter, R.C.; Knudsen K.L.; Sowers, J.M.; Wentworth, C.M.; Koehler, R.D.; and Randolph, C.E.; 2006; "Map of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Region, California"; U.S. Geological Survey Open File Report 2006-1037. According to these maps, the liquefaction susceptibility is generally classified as "Low" for the "Alluvial fan deposits (latest Pleistocene)" and "Moderate" for the "Alluvial fan deposits (Holocene)" at the project site.

Liquefaction Potential

Based on the available boring information, the subsurface soil conditions at the project site consist of mainly stiff to very stiff clay with gravel, underlain by medium dense to very dense silty/clayey sand with gravel and/or very stiff to hard lean clay.

Liquefaction potential along the project alignment was evaluated in accordance with the methods proposed by Youd (Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCCER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al. ASCE Journal of Geotechnical and Geo-environmental Engineering, October 2001, Vol. 127 No. 10). The evaluation was primarily using the boring data and a peak ground acceleration of about 0.7g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. According to Bray (2006), liquefaction appears to occur in soils where these fines are either non-plastic or are low plasticity silts and/or silty clays (PI<12%, and LL<37%), and with high water content relative to their liquid limit (w>0.85 LL).

Based on our analyses, the liquefaction potential of subsurface soil from the soil the borings along the alignment of the proposed restored pipelines is considered to be low.

Lateral Spreading

Liquefaction-induced spreading has been defined as the *"lateral displacement of large surficial blocks of soil as a result of liquefaction in a subsurface layer"*. Lateral spreading refers to the more moderate movements of gently sloping ground due to soil liquefaction. As described by Bartlett and Youd (1992a; 1992b), liquefaction-induced lateral spreading occurs on mild slopes of 0.3 to 5% underlain by loose sand and shallow water. The geologic conditions conducive to lateral spreading (gentle surface slope, shallow water table, and liquefiable cohesionless soils) are frequently found along streams and other waterfronts in recent alluvial or deltaic deposits, as well as in loosely-packed, saturated, sandy fills.

In our opinion, lateral spreading is unlikely to occur at the project site because there is no open face (water course, cliff, bank, ditch, etc.) in the vicinity of the proposed restored pipelines. There is no free surface for the potentially liquefiable soil pockets/lenses to daylight.

10.0 FINDINGS AND RECOMMENDATIONS

10.1 General

This report was prepared specifically for the proposed project. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

10.2 Earthwork and Grading

The proposed project consists of restoration of the existing pipelines. The major earthwork consists of excavation and backfilling of the trench to the original ground profile. No other grading work is required.

All grading operations should be performed in accordance with the project specifications and District's standards. A qualified Geotechnical Engineer or the District's site representative should observe the grading operation and perform moisture and density tests on prepared subgrade and compacted backfill material. Any fill material imported to the project site should be conformed to the project specifications and reviewed by the Engineer.

Should there be any alternations of the proposed construction that will affect the stated bases of our recommendations, we should be informed so that PARIKH can review such changes and amend or submit additional recommendations.

Maximum Dry Density and Optimum Moisture Content of Native Soil Modified Proctor Compaction tests (ASTM D1557) were performed on selected composite soil samples to investigate the maximum dry density and optimum moisture content of the native soil. The laboratory test results are summarized in the table below.

TABLE 5 - SUMMARY OF MODIFIED PROCTOR COMPACTION TEST RESULTS

Location	Boring Number	Depth Range (ft)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
Half Road	B-03	0-3	131.5	9.0
Half Road	B-07	0-20	135.0	7.5
Cochrane Road	B-09	0-20	127.0	9.5
Main Avenue	B-12	0-20	135.0	6.5

10.3 Settlement

Excavation and backfill for the pipeline restoration is expected to be up to about 10 feet below the existing ground surface. The subsurface soils below the assumed trench excavation generally consist of mainly dense to very dense silty/clayey sands and hard clays. These subsurface soils are considered relatively incompressible under the anticipated relatively minimal net bearing stress below the pipe invert elevation as a result of the installation of the steel pipe. Therefore settlement (both short-term and long-term) under static conditions should not be a major concern.

10.4 Recommendations

10.4.1 Modulus of Soil Reaction, E'

Flexible pipes support load from their inherent strength plus the passive resistance of the soil as the sides of the pipe deflect outward against the backfill. The supporting capability of the soil depend primarily on backfill characteristics which is represented by the modulus of soil reactions, E'.

The modulus of soil reaction (E') is not a fundamental geotechnical engineering property of the soil. This is an empirical soil-pipe system parameter depending on the deflection and the pressure developed at pipe springline during construction. For the project design, we have evaluated the research work performed by Howard (1977), Hartley and Duncan (1987), and Jeyapalan (1995).

The E' is controlled by factors including soil cover depth, pipe size, trench width, bedding type, compaction density, etc. (Jeyapalan, 1995). We have adopted the procedures proposed by Jeyapalan to establish E' for pipe design. In these procedures, the combined effect of pipe bedding material and native soils are incorporated in estimating the design modulus of soil reaction.

The following were assumed in the derivation of the E' of this project:

a) Combined effects of pipe bedding material and native soil have been incorporated in estimating design modulus of soil reaction.

b) The modulus of soil reaction of the pipe zone (E') is based on fine-grained soils with less than 35% sand content with 90% relative compaction recommended by Duncan.

c) Since the native soil can be disturbed during construction, a relatively conservative modulus of soil reaction is recommended. Based on "AWWA Manual Table 10 – Stiffness Values for In Situ (Native) Soils", the modulus of soil reaction of the native soil at the pipe invert (E_n') was estimated to be 1500 pound/inch^2 (psi) for stiff lean clay with tested undrained shear strengths between 1.3 tsf and 1.7 tsf.

d) Since the trench width (B_d) is unknown at design stage, A minimum (trench width at top of pipe (B_d)/outside diameter of pipe B_o) of 1.5 is assumed in the calculations of E'.

The recommended composite E' is shown on Plate No. 11 for different depths of soil cover.

10.4.2 Thrust Block Design and Pipe Friction

At locations where the pipeline changes direction, dead ends or develops thrusts at valves or other attachments, the tendency of movements can be resisted by (a) friction between the pipe and surrounding backfill, and (b) thrust blocks. A coefficient of 0.35 can be used to determine the friction between the pipe and the backfill.

Thrust blocks should be at least 12 inches wide. In sizing the thrust blocks for design, it should be noted that the maximum lateral pressures of the thrust block should not exceed the allowable bearing capacity of 1.8 ksf.

10.4.3 Pipe Bedding Subgrade

In general, the subsurface soil conditions encountered at the planned pipe invert depth (approximately 5 to 9.5 feet) of the restored pipelines consist of stiff to very stiff clay with gravel, underlain by medium dense to very dense silty/clayey sand with gravel and/or very stiff to hard lean clay. The subgrade should be prepared to provide uniform bearing throughout the entire length of the pipe alignment.

Areas to receive pipe bedding material should be excavated to remove any loose/soft soil materials. If weak or otherwise unsuitable soils are encountered at the base of the trench excavation and over-excavation is required, the over-excavation can be backfilled with 3/4 –inch clean, angular drained rock. The resulting surface upon which pipe bedding is to be placed should be observed by the Engineer. Areas receiving pipe bedding material should be scarified; moisture conditioned and compacted in accordance with the District specifications.

When the trench bottom is unstable due to a wet or spongy foundation, the trench bottom must be stabilized. The stabilization or backfill of the over-excavation should be referred to Section 11.3 “Working Platform”.

10.4.4 Pipe Bedding and Pipe Embedment Materials

The pipe bedding and pipe embedment materials details should conform to the District standards, which consists “Select Backfill”, “Soil Cement or Controlled Density Fill (CDF)”.

Select Backfill

The pipe bedding should consist of imported, well-graded, granular material. The select backfill shall conform to the following gradation and be within the boundary created by a straight-line interpolation between points:

Sieve Sizes	Percent Passing by Weight
¾ inch	100
No. 4	25 – 90
No. 200	0 – 12

Select backfill shall also conform to the following requirements:

Test	Test Method No. California	Individual Test Result	Moving Average
Resistance (R-value)	301	78 Minimum	-
Sand Equivalent	217	28 Minimum	31 Minimum

If Sand Equivalent exceeds 35, the R-value requirement shall be waived.

Select backfill material shall be compacted to a relative compaction of not less than 95 percent.

Based on the findings from the field exploration and laboratory soil testing, it appears that the native soil material does not meet the requirements as “Select Backfill”

Soil Cement

According to the “District Standards for Pipeline Backfill”, soil for soil cement bedding shall be either: (1) imported material which is free of organic material and other deleterious substances; or (2) commercial quality concrete sand. Imported material shall meet the following requirements:

a) The amount of soil passing the No. 200 sieve shall not exceed 15 percent, by weight, and the amount of soil passing the No. 100 sieve shall not exceed 50 percent by weight. The soil shall be non-plastic or of very low plasticity.

b) All material for the soil-cement bedding shall be selected or processed so that at least 95 percent of the material passes the No. 4 sieve and no particles are larger than 3/8 inch.

c) Clay balls in the soil shall not be allowed.

Controlled Density Fill (CDF)

According to the “District Standards for Pipeline Backfill”, controlled density fill shall consist of a mixture of aggregate, Portland cement, mineral admixtures, water, and upon the favorable review of the Engineer, chemical accelerating admixtures.

a) Portland cement shall be Type II Modified conforming to the provisions in Section 90-2.01, "Portland Cement," of the State Specifications. Mineral admixtures shall not be substituted for Portland cement.

b) Water shall conform to the provisions of Section 90-2.03, "Water," of the State Specifications. Chemical admixtures for accelerating shall be Type C or Type E conforming to the requirements in ASTM C 494.

c) Aggregate shall conform to the quality requirements of Section 90-2.02, "Aggregates," of the State Specifications. Aggregate shall be well graded from coarse to fine.

The materials anticipated within the planned depth of excavation for the proposed project are mainly very stiff to hard lean clay with sand/gravel, gravelly clay and medium dense to very dense silty/clayey sand which may not conform to the required specifications for the select backfill material. The import material should be reviewed by the Engineer prior to importing to the site.

The pipe bedding material will be generally more permeable than the surrounding native soils and may provide a conduit for the flow of groundwater. If it is desired to minimize this flow, it is our opinion that cut-off dams be placed along the pipeline at frequent intervals. The spacing of the cut-off dams is a function of the accepted risk.

The minimum thickness of pipe bedding material should be in accordance with District Standard Details ST-45 and ST-46.

The pipe shall be installed according to District's standard details as shown in Plates No. 13A through 13D. A minimum 95% relative compaction is recommended for the bedding. It is recommended that the degree of compaction be established based on density determined in accordance with ASTM D 1557. The placement, extent and relevant details of the bedding material should conform to the District's specifications and/or the pipe manufacture's recommendations.

The intermediate pipe bedding and the pipe embedment are recommended to be wrapped around by filter fabric due to the clayey nature of the native soils that exist in the foundation and if used, Type “A” backfill and Type “B” backfill above the pipe embankment section.

10.4.5 Trench Backfill

The backfill material should be conformed to the District’s specifications and may include either Type “A” backfill or Type “B” backfill.

Type “A” Backfill

Backfill material above the bedding should be Type “A” backfill per the District’s specifications and should be a minimum of 12 inch over the top of the pipeline.

According to the “District Standards for Pipeline Backfill”, Type “A” backfill may, at the Contractor’s option, be either select backfill material or suitable material from trench excavation or structure excavations from which all vegetation, Portland cement concrete, asphaltic concrete, or other debris have been removed. For material from trench or structure excavation, the material larger than ¾-inch shall not exceed 20 percent by volume and all material larger than 1½-inch shall be removed. Plastic clay with Plasticity Index (per ASTM D 4318) greater than 20 should not be used as Type “A” backfill either.

Type “A” backfill material shall be compacted in accordance with the applicable provisions of District’s specifications and to the following:

a) Within existing paved areas, sidewalks, curb and gutters, driveways, or shoulders, and within 5 horizontal feet of existing utilities, the relative compaction shall be not less than 95 percent as per ASTM D 1557.

b) Within 1-foot of the walls of all structures and related facilities, the relative compaction shall be not less than 95 percent as per ASTM D 1557.

c) In all other areas, the relative compaction shall be not less than 90 percent as per ASTM D 1557.

Based on the boring data, the subsurface soils within the planned depth of proposed trench excavation are anticipated to be mainly fine-grained lean clay/silt with sand/gravel and some clayey/silty sand with gravel. Fat clay was only encountered in Boring B-10 (middle of Cochrane Road between the Half Road/Cochrane Road intersection and E. Main Avenue/Cochrane Road intersection). Based on the subsurface soil conditions encountered in our borings, it appears that majority of the native soils will likely to meet the District's specifications for Type "A" backfill. However, further testing will be required to verify this prior to placement of any Type "A" backfill.

Type "B" Backfill

According to the "District Standards for Pipeline Backfill", Type "B" backfill material shall be clean uniformly graded material from which all vegetation, Portland cement concrete, asphaltic concrete, other debris has been removed. This material will provide a cap over the Type "A" backfill for the trench in the unpaved areas. Type "B" backfill shall have a permeability less than or equal to the native material adjacent to the trench. No particles larger than 6 inches will be allowed. Material from trench excavation or structure excavation that satisfies these requirements may be used for Type "B" backfill.

a) Type "B" backfill material shall be placed in loose layers not exceeding a thickness of 8 inches. Jetting will not be allowed.

b) Type "B" backfill material shall not be placed within 1-foot of the walls of any structures or in areas where pavement, sidewalk, curb and gutter, or shoulders are required to be removed and replaced.

c) Type "B" backfill material shall be compacted to a relative compaction of not less than 90 percent per ASTM D 1557 in the non-paved areas.

Since Type "B" backfill shall have a permeability less than or equal to the native material adjacent to the trench (which is mainly lean clay with sand/gravel). Based on the subsurface soil conditions encountered in our borings, it appears that majority of the native soils will likely to meet the District's specifications for

Type “B” backfill. However, further testing will be required to verify this prior to placement of any Type “B” backfill.

Type “A” and Type “B” Backfill

At some locations, the native clayey soil material (within the planned depth of excavation which may be used as Type “A” backfill or Type “B” backfill) may have relatively high moisture content. Saturated soils should not be placed as backfill until they are dried and processed to be closed to the laboratory optimum moisture content (as obtained from ASTM D 1557) in order to achieve the recommended relative compaction.

Alternative for Type “A” and Type “B” Backfill

“Select Backfill Material” may be used as an alternative to Type “A” backfill or Type “B” backfill.

Placement of Type “A” Backfill and Type “B” Backfill

The percent relative compaction shall be determined and controlled in accordance with the ASTM Standard Test Method D1557. The placement, extent and relevant details of the Type “A” backfill should conform to the District design standards. Field and laboratory testing of backfill materials should be performed to determine whether applicable requirements have been met.

The backfill materials should be placed by methods that do not disturb or damage the pipes. Within the pipe zone, heavy equipment operating over the pipe could result in damage, excessive deflections or other disturbances to the pipe where soil cover is insufficient. A minimum soil cover of 3 feet should be placed over the crown of the pipe prior to the use of heavy equipment. Type “A” backfill or Type “B” backfill below this level should be carefully compacted with relatively light compaction equipment to avoid overstress on the pipe.

Since the native materials are primarily fine-grained and cohesive, Jetting or flooding should not be permitted for backfill compaction.

An underground warning tape is required to be furnished and installed within the Type “A” backfill material (trench in the paved areas) or the Type “B” backfill material (trench in the unpaved areas) by the contractor.

Since the native foundation and Type “A” backfill and Type “B” backfill are generally clayey soil materials, the intermediate pipe bedding and the pipe embedment are recommended to be wrapped around by filter fabric to prevent the migration of fines. The filter fabric should be permeable and non-woven and should conform to Caltrans Standard Specification (Section 88-1.02 B).

10.4.6 Buoyancy of Pipes

Groundwater was measured at the depths between 12.0 feet and 20.3 feet in the PARIKH monitoring wells on October 15, 2015, December 24, 2015 and March 10, 2016 and the depth of 15.5 feet in the District monitoring well on December 22, 2015. The depth to the new pipe invert ranges from approximately 5 feet to 9.5 feet. The minimum soil cover ranges from approximately 2.5 feet to 7 feet for the 2.5-foot steel pipe and approximately 3 feet to 4 feet for the 3-foot steel pipe.

Groundwater level will be monitored in the installed monitoring wells during the forthcoming rainy season. The proposed restored pipelines are unlikely to be subjected to submerged or partially submerged conditions based on the current measured groundwater elevations. However, groundwater levels may fluctuate; especially the severe drought period (during the investigation) followed by the El Nino winter can significantly impact the water elevations. This should be considered in the design and construction of the proposed project and the proposed pipeline could be subjected to submerged or partially submerged conditions.

Based on our preliminary estimate with an assumed groundwater level of 10 feet below the ground surface, the cover to be provided to the restored pipelines appears to be adequate.

10.5 Shrinkage Factor for Earthwork

Shrinkage is the percentage of soil decrease in volume when it is compacted from its natural state. Shrink and bulk percentage generally have a wider range for sands and rocky, gravely soils than for clays. The excavated material from the project site is anticipated to be mostly cohesive soils such as clay and silt. Typical Shrinkage Factor of 0.9 can be assumed for the cohesive soils.

10.6 Corrosion

The corrosion investigation for this project was performed in accordance with the provisions of California Test Method 643. Chemical tests were performed on soil samples from alternate borings to evaluate the corrosion potential of the subsurface soil. A summary of the corrosion test results is presented in the following table.

TABLE 6 - SUMMARY OF CORROSION TEST RESULTS

Boring	Depth (feet)	Minimum Resistivity (ohms-cm)	pH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
B-01	5.0	2,010	6.27	23.6	33.8
B-03	5.5	3,620	7.08	14.8	23.3
B-05	5.5	4,290	7.87	13.6	26.0
B-07	6.0	1,610	7.03	12.3	7.8
B-09	7.0	800	7.66	6.5	11.8
B-11	11.0	1,020	7.56	232.9	13.2
B-13	6.0	5,230	6.94	11.9	13.9

According to Caltrans Corrosion Guidelines, November 2012 (Version 2.0), Caltrans considers a site to be corrosive to foundation element if one of the following conditions exists for the representative soil samples taken at the site:

- Chloride concentration is greater than or equal to 500 ppm,

- Sulfate concentration is greater than or equal to 2000 ppm,
- pH is 5.5 or less.

Based on the corrosion test results, the native subsurface soils in the borings at the project site are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure.

10.7 Structural Pavement Sections

New Pavement

R-value tests were conducted on representative samples in eight borings drilled in 2015 which are in the vicinity of the project site. The test results are summarized in Table 7.

TABLE 7 - SUMMARY OF R-VALUE AND PLASTICITY INDEX TEST RESULTS

Test Boring	Approx. Existing Ground Elev. (ft)	Plasticity Index	R-Value
B-02	388.5	-	43
B-03	393.5	7	-
B-04	398.0	-	20
B-05	408.0	-	< 5
B-07	423.0	-	14
B-08	473.5	-	12
B-09	450.0	16	-
B-10	424.5	-	9
B-11	414.0	-	47
B-13	394.5	7	-

Findings, Design Approach and Recommendations:

The brief summaries of the design assumptions and recommendations for the structural pavement are discussed below:

1. As shown in the "Summary of R-value and Plasticity Index Test Results", the R-values of the native basement soils were tested to be between less than 5 and 47. Considering the individual single result of R-value less than 5 and the overall test results, an R-Value of 10 was adopted for the native subgrade soil materials in the

design of structural pavement sections.

2. Based on the available boring information and laboratory test results, the pavement subgrade generally consists of lean clay with sand/gravel or gravelly clay.
3. The following Design Traffic Index (TI) was provided by the City of Morgan Hill.

Location	**Road Type**	**Design Traffic Index**
Half Road	Collector	8.0
Main Avenue	Arterial	8.0

4. The minimum pavement section will be 4-inch Asphalt Concrete (AC) over 8-inch Aggregate Base (AB) according to City of Morgan Hill Street Design Standard.
5. The recommended minimum R-value for Class 2 AB is 78.
6. Considering the relatively low R-value of the soil sample obtained at Boring B-06, it is recommended that a) minimum 6 inches of basement soil to be over-excavated; b) a layer of Subgrade Enhancement Geotextile (SEG) to be placed at the bottom of over-excavation; and c) the over-excavation to be backfilled with aggregate subbase. The lineal extent of mitigation measure will be approximately 500 feet on both sides of Boring B-06. The SEG should be conformed to Caltrans Standard Class B2.

Based on the Traffic Indices provided by the City of Morgan Hill and the design R-value, the recommended structural pavement sections are shown in the table below.

TABLE 8 – RECOMMENDED STRUCTURAL PAVEMENT SECTION

Traffic Index (TI)	**Assumed R-Value**	**Structural Pavement Section (inch)**		
		AC	**AB (2)**	**Total Thickness (inch)**
8.0	10	5	17	22

11.0 CONSTRUCTION CONSIDERATIONS

The following sections of the report include comments related to excavation, dewatering, temporary excavation and shoring, earthwork and other geotechnical aspects of the proposed construction. The sections are written primarily for the preparation of plans and specifications. Since these sections discuss potential construction issues related to the project, it may also be of use to the District's representatives involved in monitoring of construction activity. The field investigation performed by PARIKH primarily address design issues and was not intended to identify construction-related issues.

Prospective contractors for the project must evaluate construction-related issues on the basis of their own knowledge and experience in the local area, on the basis of similar projects in other localities, or on the basis of field investigation on the site performed by them, taking into account their own proposed construction methods and procedures. In addition, construction activities related to excavation and lateral earth support must conform to safety requirements of OSHA and other applicable municipal and State regulatory agencies.

11.1 Temporary Excavations Slope and Shoring

Excavations should not be expected to stand vertically without any support. According to OSHA Safety Standards, temporary excavations with personnel working within the excavations should be sloped or shored if the excavations are deeper than 5 feet. Temporary slopes may be constructed if there is no site constraint. Temporary shoring should be used if site constraints are present or at the Contractor's option. It should also be noted that the recommended temporary excavation slopes in Section 11.1.1 are for adequately dewatered excavations. All temporary excavations should be made and supported in accordance with California OSHA Safety Standards and should be designed based on the existing soil/site condition.

In our opinion, conventional equipment could be used to excavate on-site soil materials. The materials to be excavated are anticipated to be mainly very stiff to hard lean clay with sand/gravel, gravelly clay and medium dense to very dense silty/clayey sand. It is possible that unknown old buried utilities or abandoned structures, concrete rubble etc. are located along the different reaches. It might require special equipment and additional efforts to remove these buried objects.

11.1.1 Temporary Excavation Slopes

The slope height, inclination, and excavation depths should not exceed those specified in local, state, or federal safety regulations. The design of the temporary slopes by the Contractor or his specialty subcontractor should conform to the OSHA's "Guidelines for Excavations and Temporary Sloping". The Contractor or responsible subcontractor should develop their design based on the subsurface conditions exposed at the time of construction.

For excavations in homogenous soils 20 feet or less in depth, OSHA guidelines state that the maximum allowable slope should be 3/4H: 1V, 1H:1V and 1-1/2H:1V for Type A, B and C soils, respectively. Type A soils are stronger; Type B soils are intermediate, and Type C soils are weaker. Based on the available boring data, the subsurface soils are generally Type B soils. All un-shored slopes less than 20 feet deep should be excavated to inclination no steeper than 1H: 1V unless shored with applicable safety standard. It should be noted that the slope ratios recommended by OSHA are for temporary, un-surcharged slopes. Where traffic and/or heavy equipment loads are anticipated, the temporary slope should be designed for an additional uniform surcharge of 250 psf. Slopes may need to be flattened based on materials exposed during construction and any uniform surcharge at the crest of the temporary slope. Exposed slopes should be kept moist (but not saturated) during construction. The temporary cut slopes discussed above assume that the groundwater is maintained below the bottom of excavation at all time during construction.

Medium dense sand was encountered at relatively shallow depth in Borings B-06 and B-07. Medium dense sand may be encountered within the depth of excavation in the vicinity of these boring locations during construction.

Fills to be placed on temporary slope excavation should be keyed and benched into the temporary slope material. The height of the key should not be less than 3 feet or more than 5 feet and the minimum width should be 6 feet. The frequency and location of the benches may be adjusted and modified in the field as necessary.

Strength softening, sloughing and erosion could be expected for the bare surficial soil materials if the temporary slopes are exposed to weather and rain for an extended period of time. Stiff clays also tend to develop soil creep due to seasonal change in moisture content resulting in sloughing. Therefore, adequate surface protection should be provided to protect the slope surface from erosion, excessive drying and/or saturation during construction.

11.1.2 Selection of Shoring System

If the temporary slopes extend beyond the right-of-way, undercut any existing buried utilities, foundations of adjacent structures and trees (i.e. excavation is within a 1-1/2H: 1V slope projected downward from the bottom edge of adjacent footing), shoring may be necessary for the excavations. The selection, design and performance of the temporary shoring system and safety of construction personnel should be the responsibility of the Contractor or his specialty subcontractor. The Contractor should have the shoring system designed and signed by a Registered Engineer. The Contractor should evaluate the actual site conditions and select appropriate construction methods. Trenching and shoring operations should be conducted in accordance with applicable Cal OSHA codes.

For the protection of any existing utilities, active shoring system should be designed with as many supports or struts as necessary to prevent excessive straining and deformation of the supported soils. This is also important with regard to existing asphalt concrete pavements where tension cracking may develop, even under minor strains.

Different shoring systems such as soldier pile & lagging, sheet piling and speed shores can be considered as option for the shoring system. The designed shoring systems considered may include bracing system (such as “speed shoring”) with continuous support of the sidewalls, continuous interlocking sheet pile, or combination of the above depending on specific site conditions. A properly designed and constructed dewatering operation is required regardless the construction method used. Continuous interlocking sheet pile is recommended for relatively deep excavations below the groundwater level with trench bottom stability concerns. The selection of construction method, shoring system and their design and performance for the project should be the Contractor's responsibility.

The sheeting should be driven to a sufficient depth below the excavation to prevent trench bottom instability (bottom heave, piping and blow out). The required depth should be determined by the Contractor/shoring designer. The type of driving equipment (vibratory or impact hammers) employed for sheet pile installation should be chosen and verified for drivability by the shoring contractor. "Unconventional" method of driving such as banging with backhoe bucket is not recommended. The temporary sheeting should be installed on both sides of the alignment in advance of the excavation. Horizontal struts should be placed against both sides of the sheeting at regular depths as the walls are exposed in order to maintain continuous stability of the excavation. Bracings should also be installed as soon as practical against the continuous sidewall support. Failure to provide such struts/bracings in a timely manner may result in lateral creep, which may cause damage to existing facilities. Removal of sheeting should be carried out in a manner that does not adversely affect the bedding of the pipe or any other improvements. The opening after removal of sheeting should be filled with grout.

Earth Pressures for Shoring Design

Lateral earth pressure for the design of sheeting or shoring systems is shown in Plate No. 12. Any traffic and surcharge loads (such as stockpile material) should be set back at an adequate distance from the top of excavations as designed by the designer. Otherwise, these loads should be considered in the shoring design.

For a contractor-designed shoring system, the Contractor should be encouraged to verify the soil conditions in order to satisfy their design requirements and to provide adequate shoring for the project.

The recommended lateral earth pressure as discussed above is for guidance only. Lateral earth pressure should be developed as part of the shoring design to be provided by the contractor and designed and signed by a Registered Engineer.

Ground Settlement and Lateral Movement due to Shoring Deflection

Based on the paper "Construction Induced Movements of In-situ Walls" by C. Wayne Clough and Thomas D. O'Rourke, for the excavation in stiff to very hard clays, ground settlement and lateral movement due to shoring deflection is as follows:

1. The settlement of the ground surface adjacent to the shoring is on the order of 0% to 0.3% of the maximum excavation depth. Lateral movement of same order of the magnitude should also be anticipated. This assumes good workmanship and high horizontal stiffness of support to reduce lateral movement.

2. The settlement diminishes in magnitude with distance away from the shoring. Typically, the settlement diminishes in magnitude gradually to zero at a horizontal distance of about 3 times the depth.

If existing utility lines are located within the potential zone of influence, they could experience distress as a result of the movement. Refers to Section 11.5 for the discussion of protection of existing utilities.

11.2 Trench Excavation Stability

Bottom Heave

Excavation bottom instability during excavation may occur as a result of bottom heave. Bottom heave is typical for excavations in clays. Based on the boring information, the lean clay with gravel anticipated within the planned depth of excavation are generally very stiff to hard with tested undrained shear strength between 2.6 ksf and 3.4 ksf. A factor of safety of 11.0 was obtained for the stability against bottom heave. It appears that the stability against the bottom heave should not be a concern.

Blow-out

Groundwater was measured at the depths between 12.0 feet and 20.3 feet in the monitoring wells in October, 2015. Potential for "Blow-out" condition at the base of the trench was evaluated in case artesian water pressures exist below an open excavation. While the depth of the trench is planned to be 10 feet, a trench depth of 11 feet was used in the analyses to account for 12 inches of over-excavation if soft/loose soils conditions are encountered at the bottom of planned excavation. With assumed groundwater depth of 10 feet below existing ground surface during the rainy season, a minimum factor of safety of 1.5 against "blow-out" was obtained based on the analyses. The minimum factor of safety required is 1.5.

The potential for bottom heave and/or "blow-out" should be monitored during the trench excavation. Deep shoring system and construction dewatering may be required to mitigate the condition.

11.3 Working Platform

If groundwater is encountered during excavation, soft and loose, saturated native soil deposits may be encountered at the trench/excavation bottom. In such cases working conditions at the trench/excavation bottom may become difficult; equipment used at the bottom of the trench/excavation may lose mobility, etc. The Contractor should take adequate measures to minimize the disturbance of the sensitive deposits at the bedding/excavation subgrade.

The Contractor may minimize the disturbance of sensitive deposits or mitigate existing soft ground conditions by constructing a working platform at the bottom of the trench/excavation. The working platform may be installed by

(a) Over-excavating minimum 12 inches below the planned bedding subgrade. The actual depth of over-excavation needed will depend on the field conditions and should be evaluated by the geotechnical engineer on a case-by-case bases during construction;

(b) Place either polyester or polypropylene subgrade enhancement geotextile (Caltrans Standard Specifications Section 88-1.02O Class B2) at the bottom of the resulting excavation;

(c) Backfill with ¾ inch angular drain rock or other such approved bridging material. The drain rock should be hard, durable, clean gravel and shall be free from organic material, clay balls or other deleterious substances; and

(d) Overlapping the ends of the subgrade enhancement geotextile on top of the drain rock or other bridging material for a minimum distance of 2 feet. The imported drain rock or other bridging material should be approved by the Geotechnical Engineer prior to importing and placing the materials on the project site. The top of the bridging layer should be densified by vibratory compaction equipment. The Contractor's method of compacting the bridging layer should be reviewed by the Geotechnical Engineer.

The contractor may use other methods of subgrade stabilization. The contractor's proposed method should be reviewed by the Engineer.

11.4 Dewatering

As described in the Section "Subsurface Soil Conditions", groundwater was measured at the depths between 12.0 feet and 20.3 feet in the PARIKH monitoring wells in October, December 2015 and March 2016 and in District monitoring well in December 2015. Based on our borings and monitoring well readings along the pipeline alignment, groundwater should not be anticipated during excavation. However, the subsurface soil conditions consist of mainly clay with sand/gravel underlain by medium dense to very dense sand/gravel in some of the soil borings, perched water table may be encountered if excavation is performed in winter or early spring. Localized dewatering should be less likely if the excavation is performed during the summer months.

Groundwater may be fluctuate especially the severe drought period (during the investigation) followed by the El Nino winter can significantly impact the water elevations. Groundwater should be verified by the contractor before construction and monitored during construction. A properly designed and constructed dewatering operation may be required. The groundwater should be maintained at least 3 feet below the bottom of the excavation at all times.

All dewatering schemes proposed by the contractor should be submitted to the District prior to implementation. However, developing and implementing an effective dewatering program should be the contractor's responsibility. A properly designed and constructed dewatering operation is recommended irrespective of the construction method used. This may include, but not limited to, continuous inter-locking sheet piles, deep soil mix walls or other appropriate methods. The contractor should install a dewatering system that will maintain the groundwater level below the bottom of the excavation at all times. In some areas that have inflow water conditions, which are difficult to effectively dry the bottom of the excavation, it may be necessary to maintain the groundwater at a deeper depth.

All dewatering systems should be properly designed to prevent pumping soil fines with the discharge water. The contractor should sample and test the groundwater for soil fine content from the wells, as needed. If soil fines are being pumped, the Contractor should revise his dewatering operations; otherwise, it may result in failure of shoring, partial instability of excavation bottom resulting in intolerable settlement of foundations of existing structures and utilities and unsafe working conditions. The Contractor should provide discharge sampling locations for each pump. Prior to excavation, the Contractor should install observation wells at frequent locations along the alignment of the reach, to verify and demonstrate to the District that the required drawdown has been attained. Contractor is encouraged to perform their own investigation, test program, etc. prior to construction in order to satisfy their design requirements for an effective dewatering program.

11.5 Protection of Existing Utilities

According to the District, there are two 34-inch High Pressure Gas Transmission Lines [cast iron or steel pipe according to Pacific Gas & Electric (PG&E)] at 4 different locations along the alignment of the restored pipelines.

All underground utilities, such as sewer laterals, water services, gas services and underground electrical or telephone conduits crossing the trench line should be located and exposed if necessary ahead of trenching operations. All underground utilities within the limits of work shall be protected from damage due to construction related activities. Submission of "Utility Protection Plan" is recommended prior to the construction with the following minimum requirements:

a) The utility protection plan should be submitted before the beginning of the construction work.

b) Include the following details:

   - Location of all utilities impacted by proposed construction by type, size, and status of use (i.e. whether in-use, abandoned, or other designation).
   - Methods employed by the contractor to locate each and every utility.
   - List of utility owners in the construction site area and the 24-hour emergency contact numbers for those utilities.
   - Schematic drawing(s) and written description of methods planned by the contractor to protect and maintain in-service utilities during construction, including initial excavation through final backfill.
   - A plan for compacting backfill under exposed water facilities.

c) For trench excavation near adjacent utilities, an active shoring system (i.e. braced shoring such as "speed shoring") can be used to minimize any potential creep movement of the soil behind the shoring. Rotational movement of the supporting soils behind the passive shoring system (i.e. sheet pile wall or soldier pile and lagging wall) can occur and potentially damage the adjacent utilities. An active shoring or shielded shoring backfilled between the trench wall and the shoring box should be used for the protection of existing utilities of this project.

d) All sewer and utility lines that cross or lie along the trench shall be adequately supported during construction and such support left in place. Care should be exercised when backfilling around such lines to avoid any damage to these utilities.

e) Any damage done to any existing utilities should be reported to the respective utility owner by the contractor.

11.6 Trench Widths

All trenches should be of sufficient width to provide free working space on either side of the applicable installation. Trench width can affect the level of compaction which is important for proper installation and its performance such as:

i) Actual compaction levels can be affected. Compaction levels that are lower than specified can lead to an under-design. The modulus of soil reaction is a function of soil type and compaction level.

ii) The width of trench can be the controlling factor in determining the load exerted on the pipe under the trench condition and embankment (negative projecting) condition.

The following are the impacts on the compaction level for "wider than specified trench width" and "narrower than specified trench width":

Wider than Specified Trench Width

a) A wider trench width creates more compactive surface area and hence a higher degree of variability can exist in achieving the proper compaction. This can lead to lower than specified compaction levels.

b) Wider than unnecessary trench widths can add to project cost due to unnecessary excavation, unnecessary imported material costs and additional placement and compaction efforts.

Narrower than Specified Trench Width

c) Trench widths that are too narrow can limit the use of proper compaction equipment and hence ability to achieve proper compaction levels.

According to California Amendments to AASHTO LRFD Bridge Design Specifications – Sixth Edition Section 12: Buried Structures and Tunnel Liners, the minimum trench width shall

a) Provide a 24-inch minimum side wall clearance between the pipe and the trench wall to ensure sufficient working room to properly and safely place compact backfill material.

b) Narrow trench widths may be used if specially designed equipment enable satisfactory installation and embedment provided an installation meeting the requirement of the project specifications and the use as approved by the Engineer.

The trench width should be prepared in accordance with the pipe manufacturer's recommendations.

11.7 Construction Monitoring

All excavations should be closely monitored during excavation/construction to detect any evidence of instability, soil creep, settlements, etc. Appropriate mitigation measures and a comprehensive monitoring plan should be implemented to correct such situations that may cause or lead to future damage to residence/improvements, existing utilities and streets. The project specific crack survey and construction monitoring program should be implemented and is critical to this project. This can alert the contractor and/or the District to minimize potential risks and to allow implementation of mitigation measures (if any).

12.0 PLAN REVIEW

This report is prepared for the proposed "Main Avenue and Madrone Pipelines Restoration Project". We recommend that final plans for the proposed project to be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred. However, design-build elements should be reviewed only from overall compliance standpoint.

13.0 CONSTRUCTION OBSERVATION

To a degree, the performance of any structure is dependent upon construction procedures and quality control measures. Hence, geotechnical observation and testing of grading operations, and foundation excavations should be carried out by the Geotechnical Engineer. If the subsurface conditions different from those forming the basis of our recommendations are encountered, this office should be informed in order to assess the need for design changes. Therefore, the recommendations presented in this report are contingent upon good quality control and these geotechnical observations during construction.

14.0 INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on our site reconnaissance and the assumption that the subsurface conditions do not deviate from observed conditions. All work done is in accordance with generally accepted geotechnical engineering principles and practices. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings. The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater or air, below or around this site. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

This report has been prepared for the proposed project as described earlier, to assist the engineer in the design of this project. In the event any changes in the design or location of the facilities are planned, or if any variations or undesirable conditions are encountered during construction, our conclusions and recommendations shall not be considered valid unless the changes or variations are reviewed and our recommendations modified or approved by us in writing.

This report is issued with the understanding that it is the designer's responsibility to ensure that the information and recommendations contained herein are incorporated into the project and that necessary steps are also taken to see that the recommendations are carried out in the field.

The findings in this report are valid as of the present date. However, changes in the subsurface conditions can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or from the broadening of knowledge. Accordingly, the findings in this report might be invalidated, wholly or partially, by changes outside of our control.

Respectfully submitted,
PARIKH CONSULTANTS, INC.

Alston Lam, P.E., G.E. 2605	Y. David Wang, Ph.D., P.E. 52911
Project Engineer	Project Manager

S:\Ongoing\Alston Lam\2014-101-TO3 Main Avenue and Madrone Pipeline Restoration Project\Report\Words\Main Avenue and Madrone Pipeline Restoration Project (Final).rpt (4-2016).doc

REFERENCES

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4. Bartlett, S.F. & Youd, T.L., 1992a. Empirical prediction of lateral spread displacement., Proc., 4th Japan-U.S. Work-shop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction, Tech. Rep. NCEER-92-0019, August 12, pp. 351–365.
5. Bartlett, S.F. & Youd, T.L., 1992b. Empirical analysis of horizontal ground displacement generated by liquefaction-induced lateral spreads., Tech. Rep. NCEER-92-0021, National Center for Earthquake Engineering Research, Buffalo, NY, August 17.
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15. Howard, A.K, 1977, Modulus of Soil Reaction Values for Buried Flexible Pipe, Journal of the Geotechnical Engineering Division, Vol. 103, No. 1, January 1977, pp. 33-43.

16. Jeyapalan, J.K., Watkins, R., 2005, Closure to ‘Modulus of Soil Reaction (E’) Values for Pipeline Design’, Journal of Transportation Engineering, Vol. 131, No. 8, August 2005, pp. 644-644.
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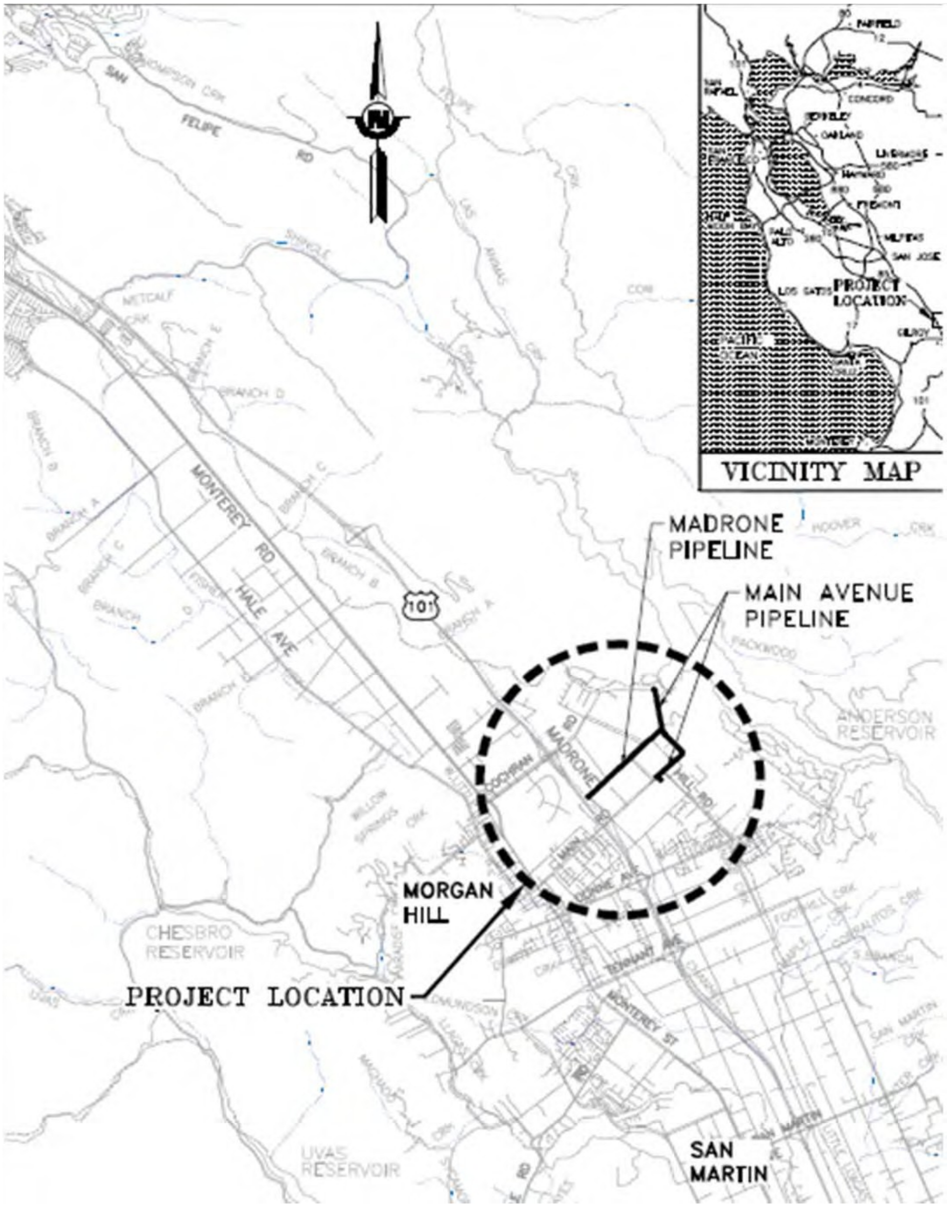
19. California Amendments to AASHTO LRFD Bridge Design Specifications – Sixth Edition Section 12: Buried Structures and Tunnel Liners.
20. C. Wayne Clough and Thomas D. O’Rourke, Construction Induced Movements of In-situ Walls.

SOURCE:

Provided by Santa Clara Valley Water Distrcit



VICINITY MAP

SCALE

0 5000 ft

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

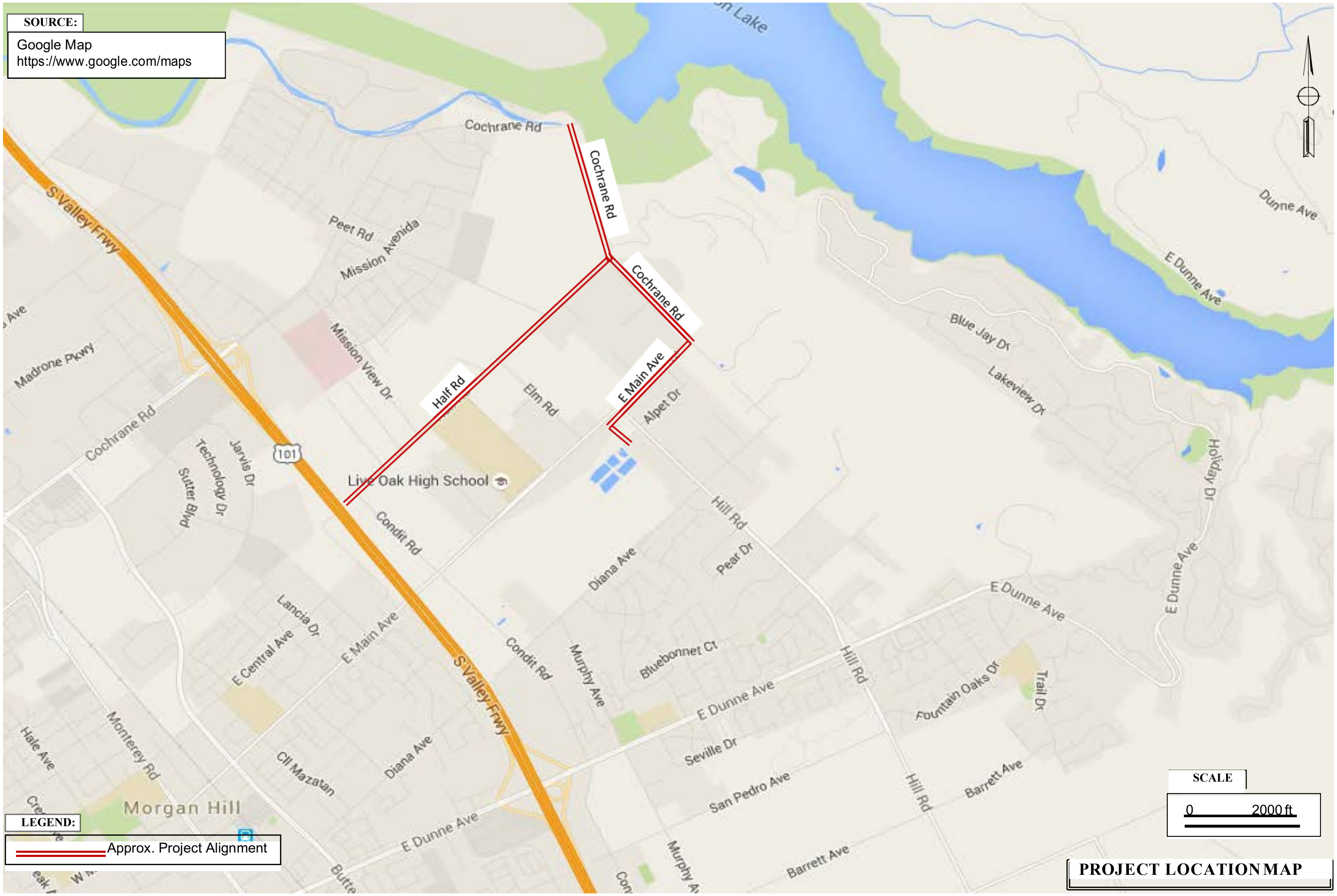
MAIN AVENUE AND MADRONE PIPELINES RESTORATION
MORGAN HILL, CALIFORNIA

JOB NO.: 2014-101-TO3

PLATE NO.: 1

SOURCE:

Google Map
https://www.google.com/maps



LEGEND:

Approx. Project Alignment

PROJECT LOCATION MAP

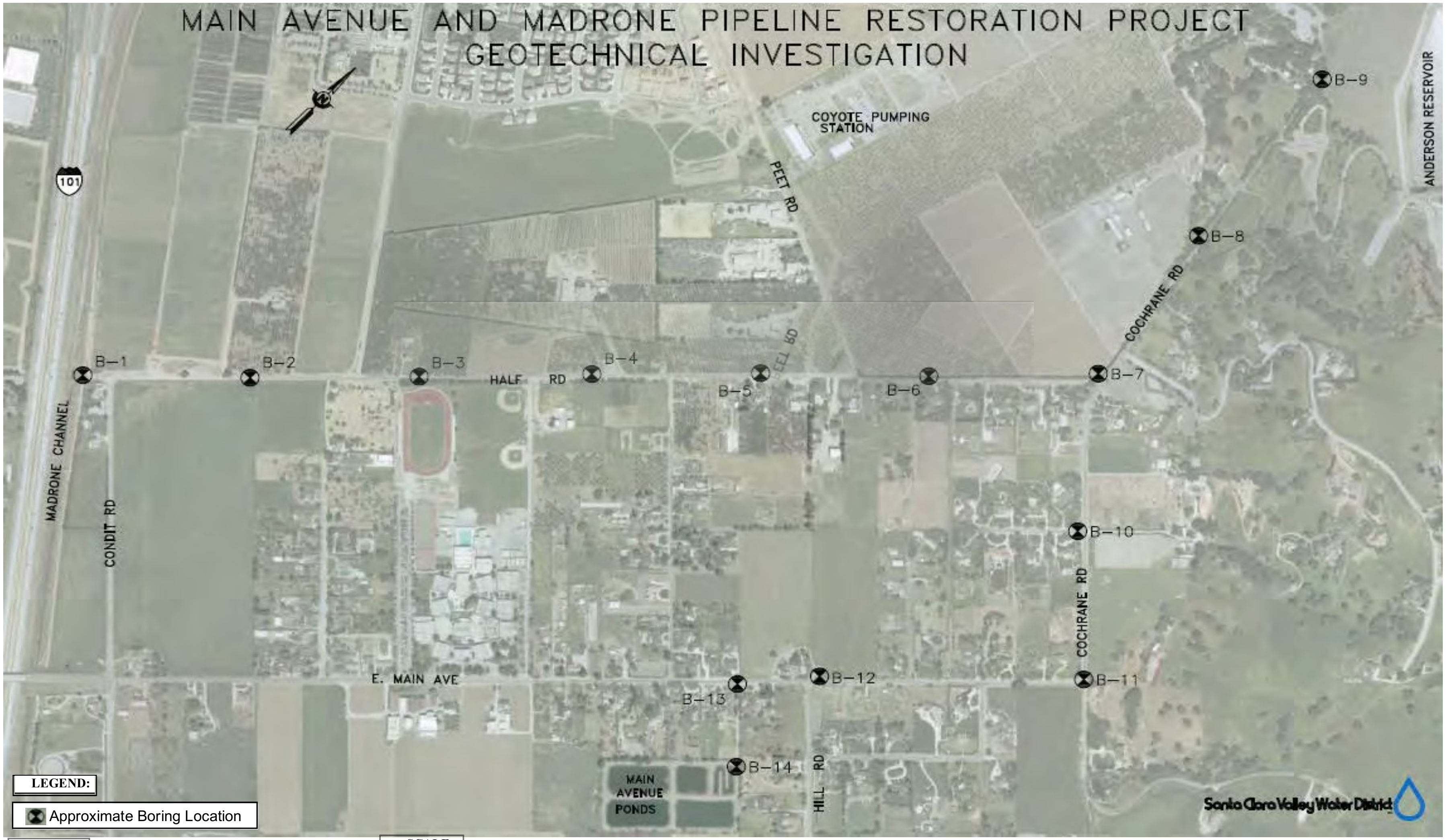
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

MAIN AVENUE AND MADRONE PIPELINES RESTORATION
MORGAN HILL, CALIFORNIA

JOB NO.: 2014-101-TO3

PLATE NO.: 2

MAIN AVENUE AND MADRONE PIPELINE RESTORATION PROJECT
GEOTECHNICAL INVESTIGATION



LEGEND:

● Approximate Boring Location

SOURCE:

Provided by Santa Clara Valley Water District

SCALE

0 1 000ft

BORING LOCATION MAP

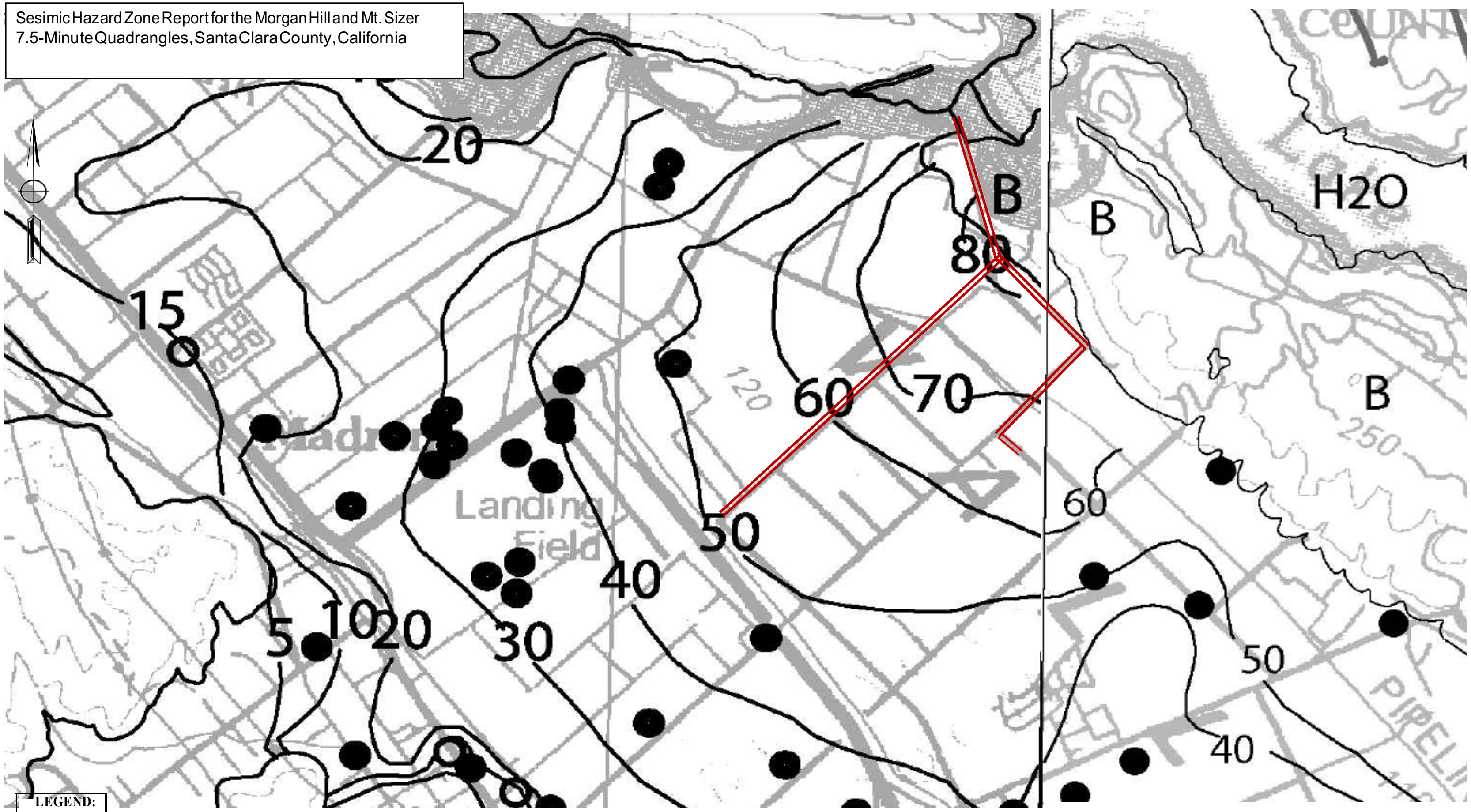
PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

MAIN AVENUE AND MADRONE PIPELINES RESTORATION
MORGAN HILL, CALIFORNIA

JOB NO.: 2014-101-TO3	**PLATE NO.: 3**

SOURCE:

Sesimic Hazard Zone Report for the Morgan Hill and Mt. Sizer
7.5-Minute Quadrangles, Santa Clara County, California



LEGEND:

▬	Approx. Project Alignment
●	Geotechnical borings used in liquefaction evaluation
○	Santa Clara Valley Water District ground-water level measurements
B = Pre-Quaternary bedrock.	
50	Depth to ground water, in feet

SCALE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

HISTORICALLY HIGH GROUND WATER

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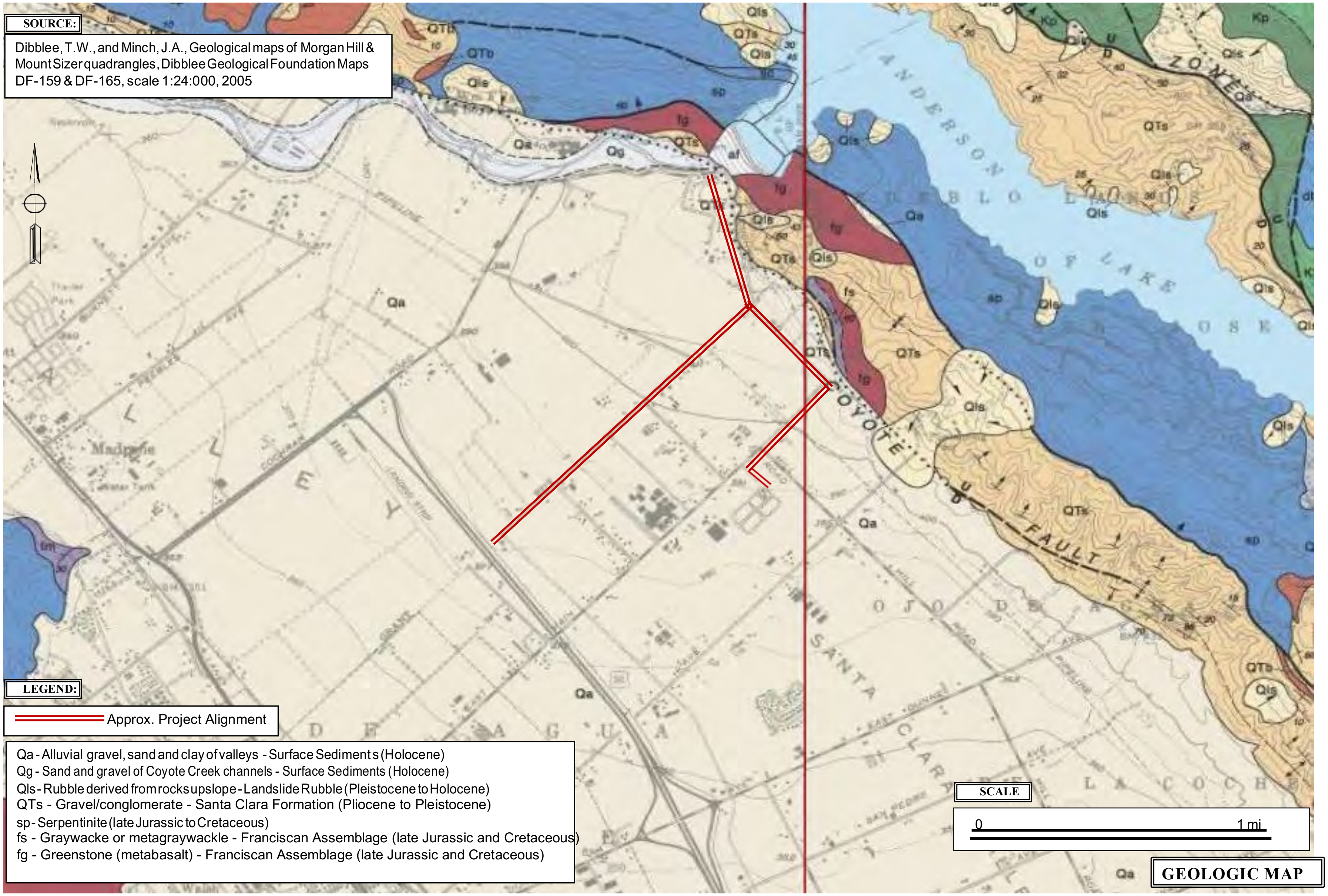
MATERIALS TESTING

MAIN AVENUE AND MADRONE PIPELINES RESTORATION
MORGAN HILL, CALIFORNIA

JOB NO.: 2014-101-TO3

PLATE NO.: 4



SOURCE:

Dibblee, T.W., and Minch, J.A., Geological maps of Morgan Hill & Mount Sizer quadrangles, Dibblee Geological Foundation Maps DF-159 & DF-165, scale 1:24:000, 2005

LEGEND:

Approx. Project Alignment

Qa - Alluvial gravel, sand and clay of valleys - Surface Sediments (Holocene)
Qg - Sand and gravel of Coyote Creek channels - Surface Sediments (Holocene)
Qls - Rubble derived from rocks upslope - Landslide Rubble (Pleistocene to Holocene)
QTs - Gravel/conglomerate - Santa Clara Formation (Pliocene to Pleistocene)
sp - Serpentinite (late Jurassic to Cretaceous)
fs - Graywacke or metagraywackle - Franciscan Assemblage (late Jurassic and Cretaceous)
fg - Greenstone (metabasalt) - Franciscan Assemblage (late Jurassic and Cretaceous)

SCALE

0 1 mi

GEOLOGIC MAP

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GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

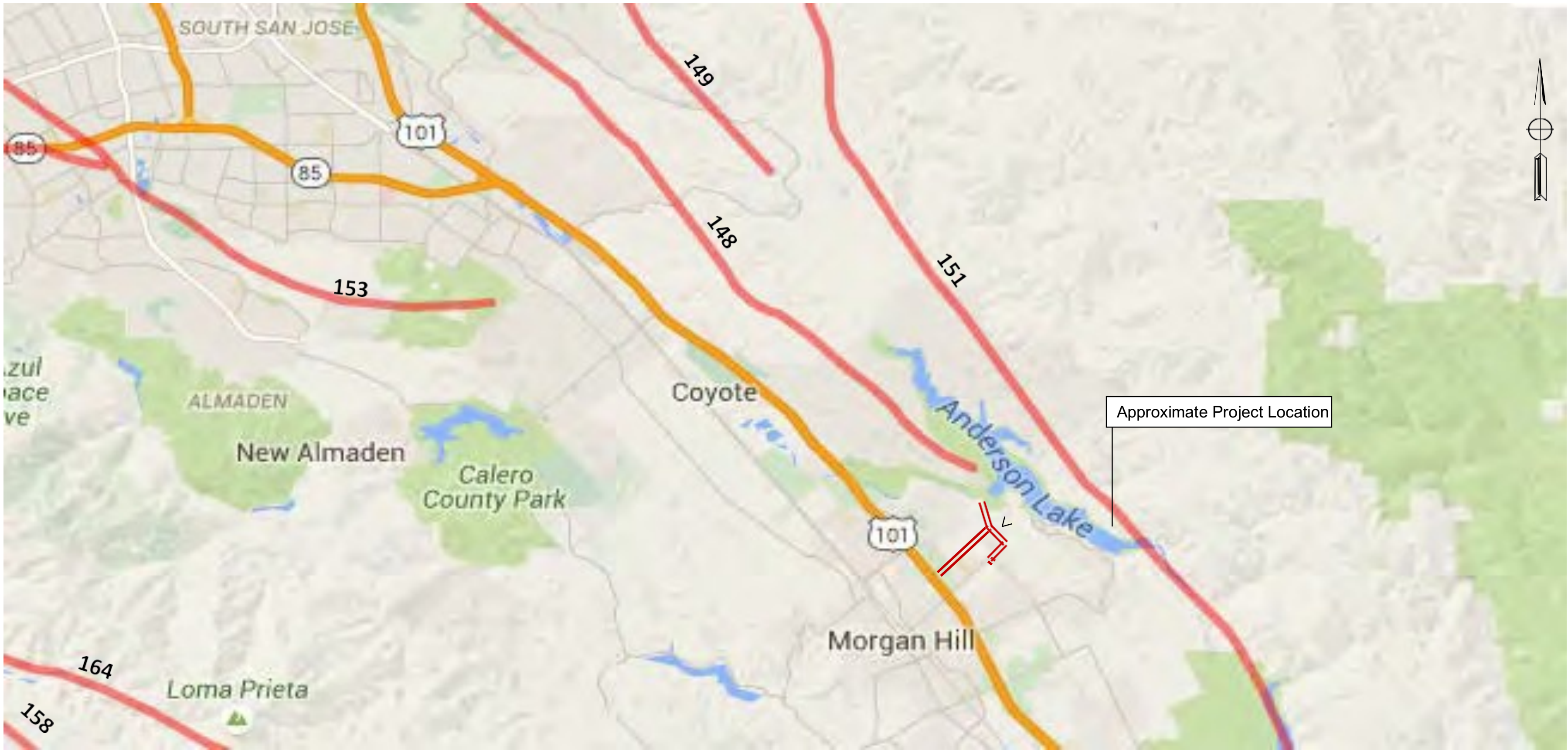
MAIN AVENUE AND MADRONE PIPELINES RESTORATION
MORGAN HILL, CALIFORNIA

JOB NO.: 2014-101-TO3

PLATE NO.: 5

SOURCE:

Source: Caltrans ARS Online Web Site
http://dap3.dot.ca.gov/ARS_Online/



LEGEND:

148 - Silver Creek - Mmax = 6.9
149 - Hayward (Southern extension) - Mmax = 6.7
151 - Calaveras (Central) 2011 CFM - Mmax = 6.9
153 - Cascade fault - Mmax = 6.7
158 - San Andreas (Santa Cruz Mts) 2011 CFM
164 - Sargent fault (northwestern section)

SCALE

0 5 mi

FAULT MAP

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MATERIALS TESTING

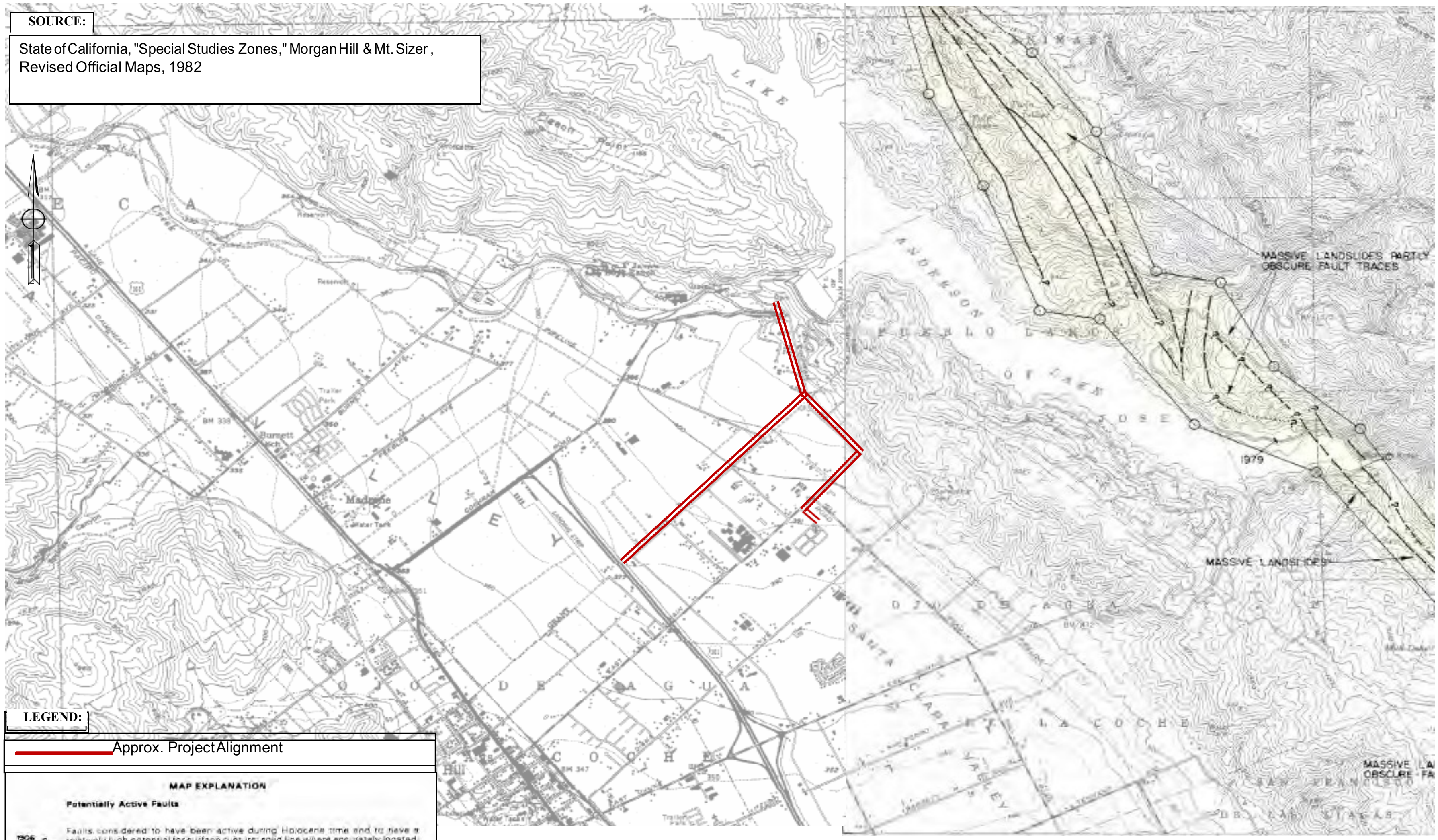
MAIN AVENUE AND MADRONE PIPELINES RESTORATION
MORGAN HILL, CALIFORNIA

JOB NO.: 2014-101-TO3

PLATE NO.: 6

SOURCE:

State of California, "Special Studies Zones," Morgan Hill & Mt. Sizer , Revised Official Maps, 1982



LEGEND:

Approx. Project Alignment

MAP EXPLANATION

Potentially Active Faults

1906 C

Faults considered to have been active during Holocene time and to have a relatively high potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

Special Studies Zone Boundaries

These are delineated as straight-line segments that connect encircled turning points so as to define special studies zone segments.

Seaward projection of zone boundary.

SCALE

0 1 mi

STATE SPECIAL STUDIES ZONES MAP (FAULT)

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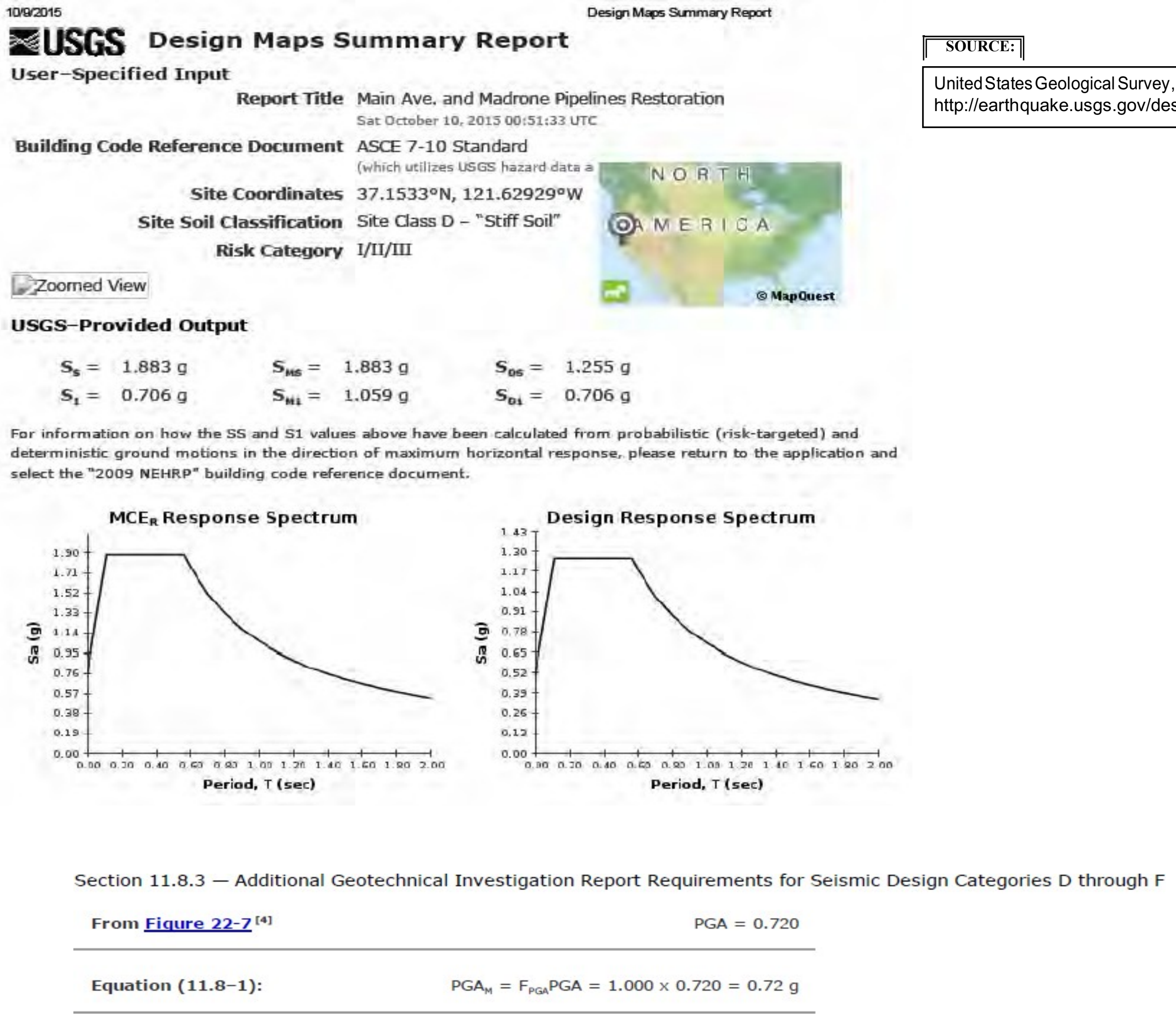
MATERIALS TESTING

MAIN AVENUE AND MADRONE PIPELINES RESTORATION
MORGAN HILL, CALIFORNIA

JOB NO.: 2014-101-TO3

PLATE NO.: 7



10/9/2015 Design Maps Summary Report

USGS **Design Maps Summary Report**

User–Specified Input

Report Title	Main Ave. and Madrone Pipelines Restoration Sat October 10, 2015 00:51:33 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data a
Site Coordinates	37.1533°N, 121.62929°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

Zoomed View

USGS–Provided Output

S_S = 1.883 g S_{MS} = 1.883 g S_{DS} = 1.255 g

S_1 = 0.706 g S_{M1} = 1.059 g S_{D1} = 0.706 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE$_R$ Response Spectrum

Design Response Spectrum

Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4]	PGA = 0.720
Equation (11.8–1):	$PGA_M = F_{PGA}PGA = 1.000 \times 0.720 = 0.72$ g

SOURCE:

United States Geological Survey, "U.S. Seismic Design Maps" Website, http://earthquake.usgs.gov/designmaps/us/application.php

CBC ACCELERATION RESPONSE SPECTRA

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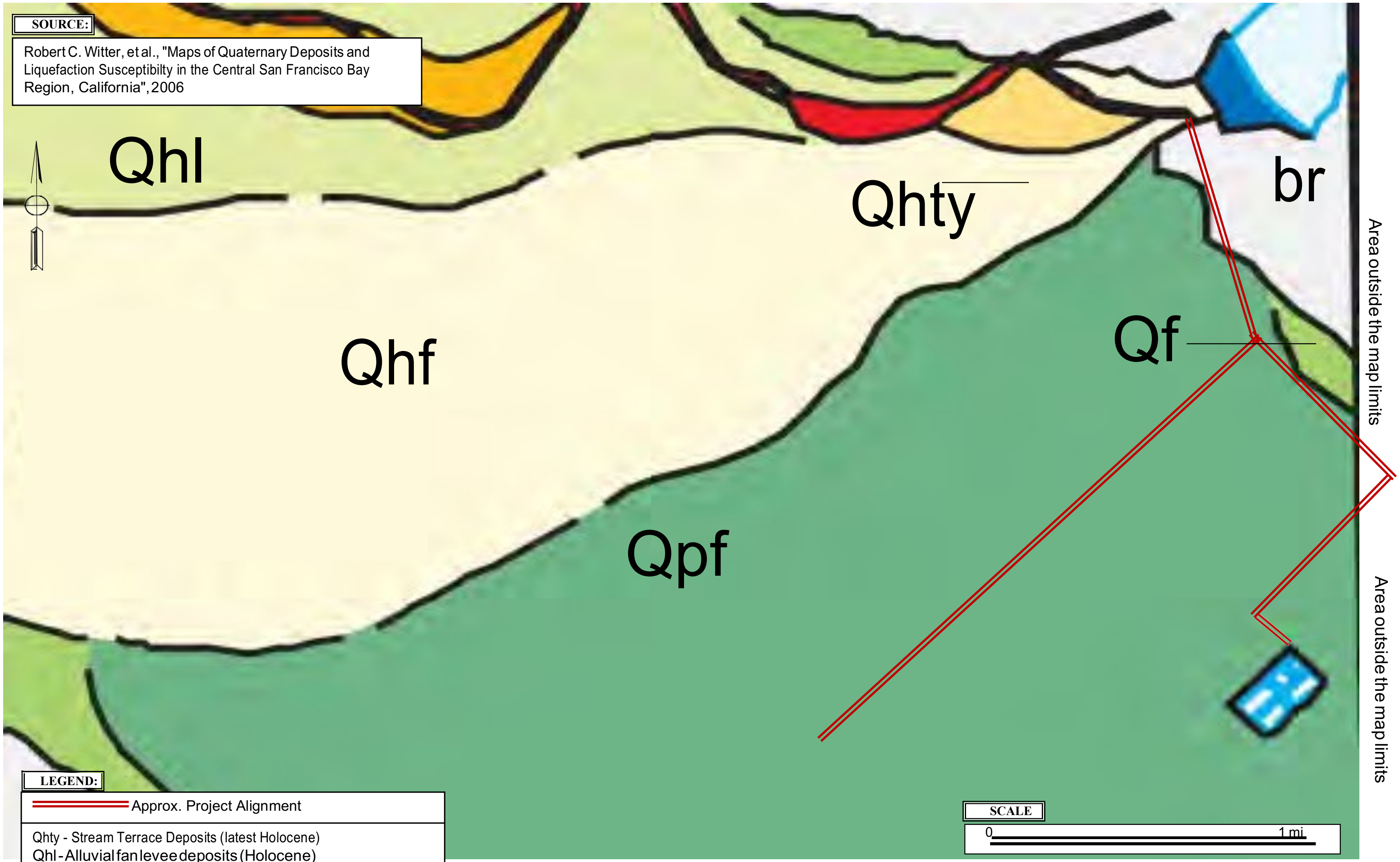
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

MAIN AVENUE AND MADRONE PIPELINES RESTORATION
MORGAN HILL, CALIFORNIA

JOB NO.: 2014-101-TO3

PLATE NO.: 8



SOURCE:

Robert C. Witter, et al., "Maps of Quaternary Deposits and Liquefaction Susceptibilty in the Central San Francisco Bay Region, California", 2006

LEGEND:

🟥 Approx. Project Alignment

Qhty - Stream Terrace Deposits (latest Holocene)
Qhl - Alluvial fan levee deposits (Holocene)
Qhf - Alluvial fan deposits (Holocene)
Qf - Alluvial fan deposits (Holocene to latest Pleistocene)
Qpf - Alluvial fan deposits (latest Pleistocene)
br - Older deposits and bedrock

SCALE

0 1 mi

QUATERNARY DEPOSITS MAP

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MATERIALS TESTING

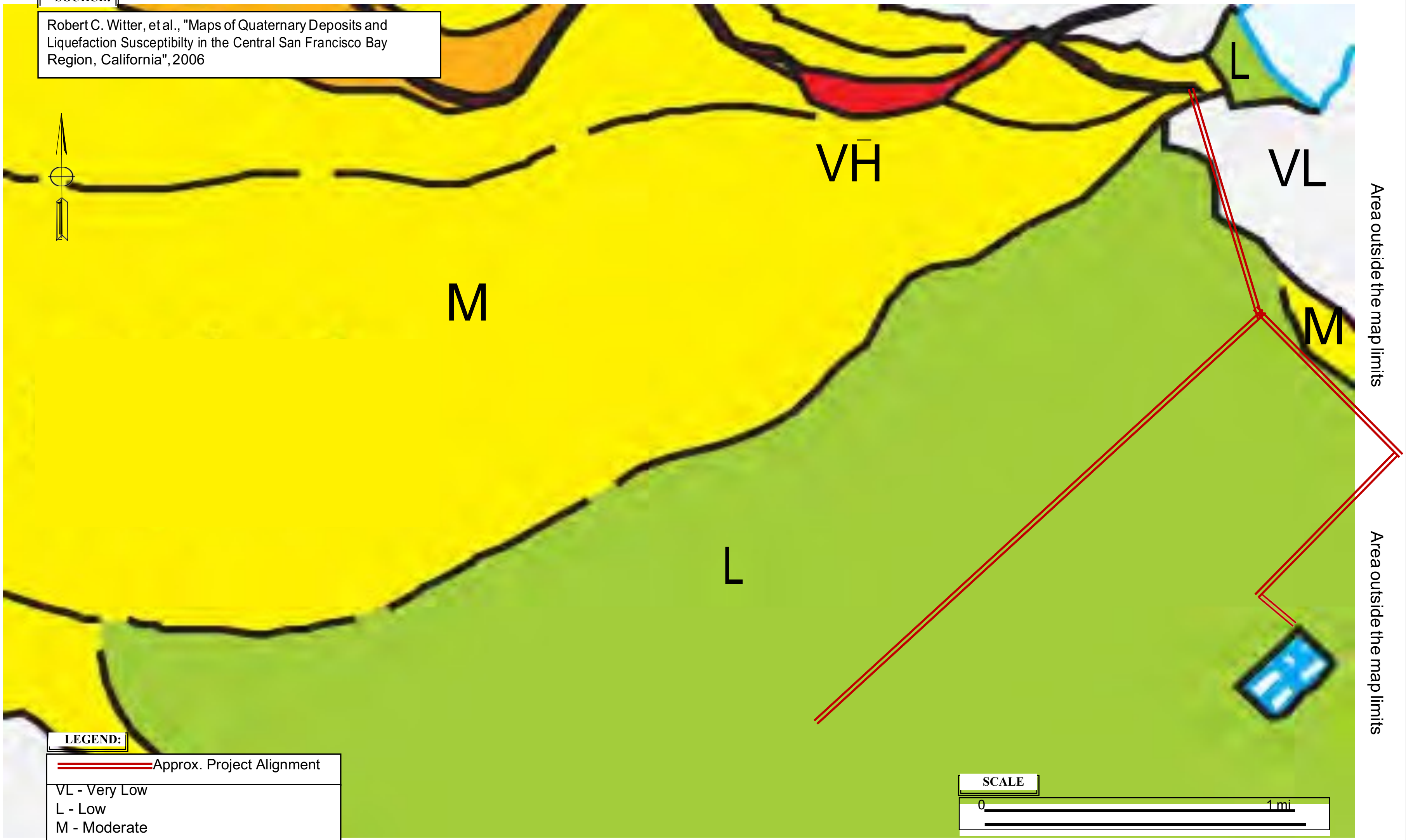
MAIN AVENUE AND MADRONE PIPELINES RESTORATION
MORGAN HILL, CALIFORNIA

JOB NO.: 2014-101-TO3

PLATE NO.: 9

SOURCE:

Robert C. Witter, et al., "Maps of Quaternary Deposits and Liquefaction Susceptibilty in the Central San Francisco Bay Region, California", 2006



LEGEND:

═ Approx. Project Alignment

VL - Very Low
L - Low
M - Moderate
H - High
VH - Very High

LIQUEFACTION SUSCEPTIBILITY MAP

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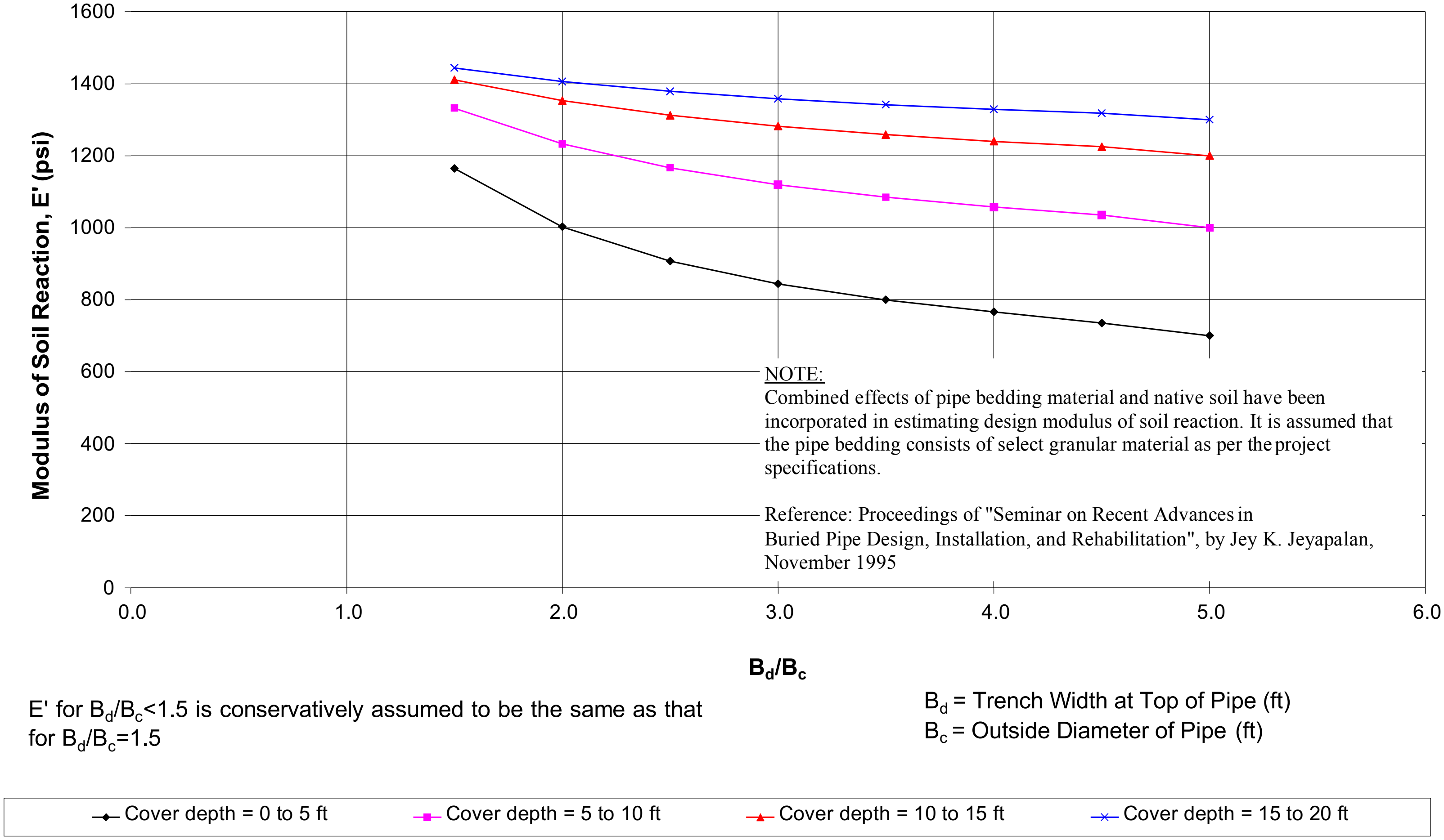
MATERIALS TESTING

MAIN AVENUE AND MADRONE PIPELINES RESTORATION
MORGAN HILL, CALIFORNIA

JOB NO.: 2014-101-TO3

PLATE NO.: 10

DESIGN MODULUS (E') OF SOIL REACTION OF SOIL AT SIDES OF PIPE
(Based on 90% Compaction)



NOTE:
Combined effects of pipe bedding material and native soil have been incorporated in estimating design modulus of soil reaction. It is assumed that the pipe bedding consists of select granular material as per the project specifications.

Reference: Proceedings of "Seminar on Recent Advances in Buried Pipe Design, Installation, and Rehabilitation", by Jey K. Jeyapalan, November 1995

E' for B_d/B_c<1.5 is conservatively assumed to be the same as that for B_d/B_c=1.5

B_d = Trench Width at Top of Pipe (ft)
B_c = Outside Diameter of Pipe (ft)

*Modulus of Soil Reaction E_n' for the Native Soil Assumed to be 1500 psi

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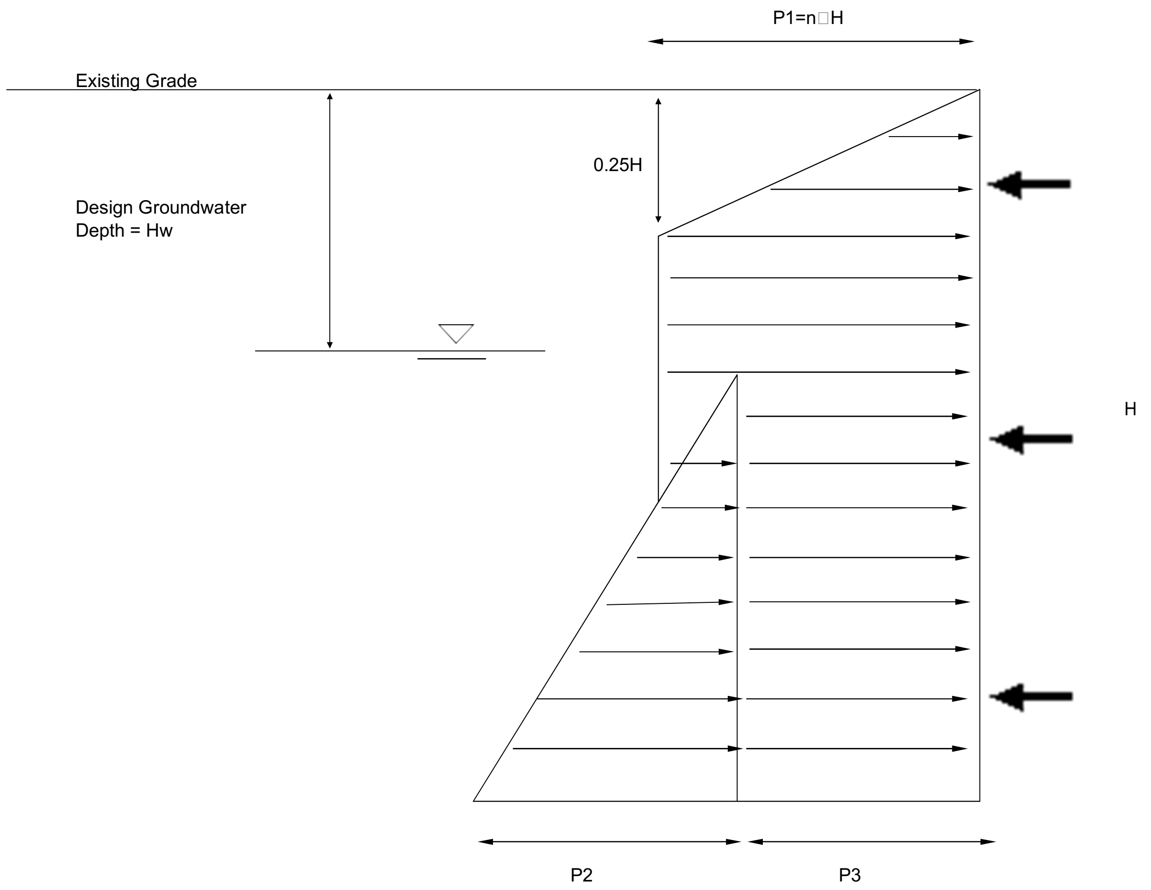
MATERIALS TESTING

MAIN AVENUE AND MADRONE PIPELINES RESTORATION
MORGAN HILL, CALIFORNIA

JOB NO.: 2014-101-TO3

PLATE NO.: 11



H = Excavation Depth in feet
P1 = n□□ = 0.3(120)H = 36 H psf
P2 = 62.4 (H-Hw) psf
P3 = n(□Hw + □'(H - Hw)) psf

n= 0.3
□ = 120 pcf (Total Unit Weight)
□' = 58 pcf (Buoyant Unit Weight)

1. The pressure diagram is based on average soil and groundwater conditions.
2. Lateral earth pressure should be checked where conditions are different.
3. All surcharge loads due to traffic and construction operation should be added for designing the shoring system.

APPARENT LATERAL EARTH PRESSURE DIAGRAM

PARIKH CONSULTANTS, INC.
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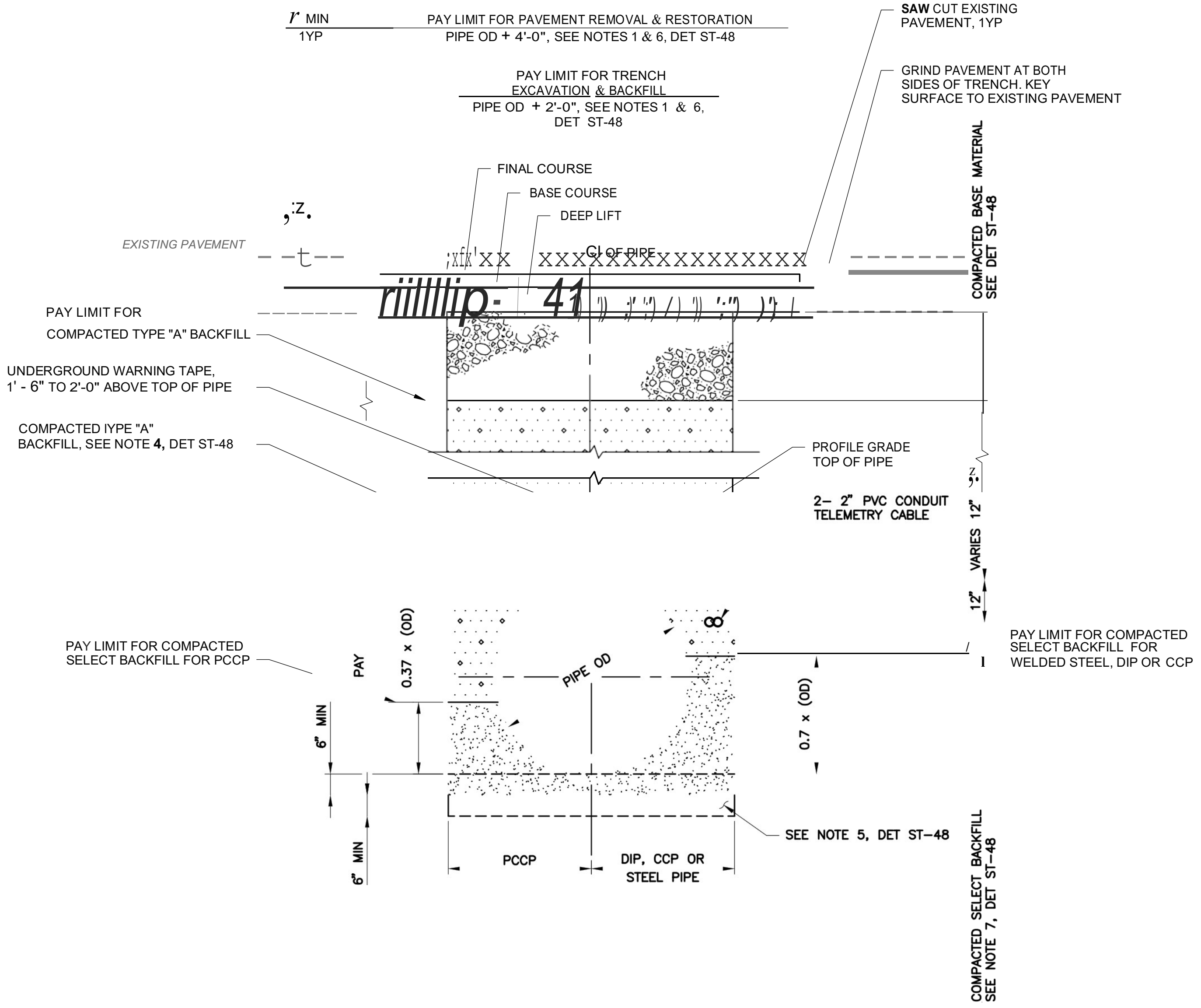
MATERIALS TESTING

MAIN AVENUE AND MADRONE PIPELINES RESTORATION
MORGAN HILL, CALIFORNIA

JOB NO.: 2014-101-TO3

PLATE NO.: 12


r MIN
1YP
PAY LIMIT FOR PAVEMENT REMOVAL & RESTORATION
PIPE OD + 4'-0", SEE NOTES 1 & 6, DET ST-48
SAW CUT EXISTING PAVEMENT, 1YP
GRIND PAVEMENT AT BOTH SIDES OF TRENCH. KEY SURFACE TO EXISTING PAVEMENT
PAY LIMIT FOR TRENCH EXCAVATION & BACKFILL
PIPE OD + 2'-0", SEE NOTES 1 & 6, DET ST-48
FINAL COURSE
BASE COURSE
DEEP LIFT
EXISTING PAVEMENT
CL OF PIPE
COMPACTED BASE MATERIAL SEE DET ST-48
PAY LIMIT FOR COMPACTED TYPE "A" BACKFILL
UNDERGROUND WARNING TAPE, 1' - 6" TO 2'-0" ABOVE TOP OF PIPE
COMPACTED IYPE "A" BACKFILL, SEE NOTE 4, DET ST-48
PROFILE GRADE TOP OF PIPE
2- 2" PVC CONDUIT TELEMETRY CABLE
12" VARIES 12"
12"
PAY LIMIT FOR COMPACTED SELECT BACKFILL FOR PCCP
PAY
0.37 x (OD)
PIPE OD
PAY LIMIT FOR COMPACTED SELECT BACKFILL FOR WELDED STEEL, DIP OR CCP
0.7 x (OD)
6" MIN
6" MIN
SEE NOTE 5, DET ST-48
PCCP
DIP, CCP OR STEEL PIPE
COMPACTED SELECT BACKFILL SEE NOTE 7, DET ST-48

t;;JIYP-45 **TYPICAL SECTION - TRENCH AT PAVED AREAS**

SCALE: NTS

Plate No. 13A

Plans\Revised Details_2015-04\ST-45.dwg

DATE	OCT 2014
DESIGN	X.X.X
DRAWN	J. FORD

PROJECT NAME AND SHEET DESCRIPTION:

STANDARD DETAILS

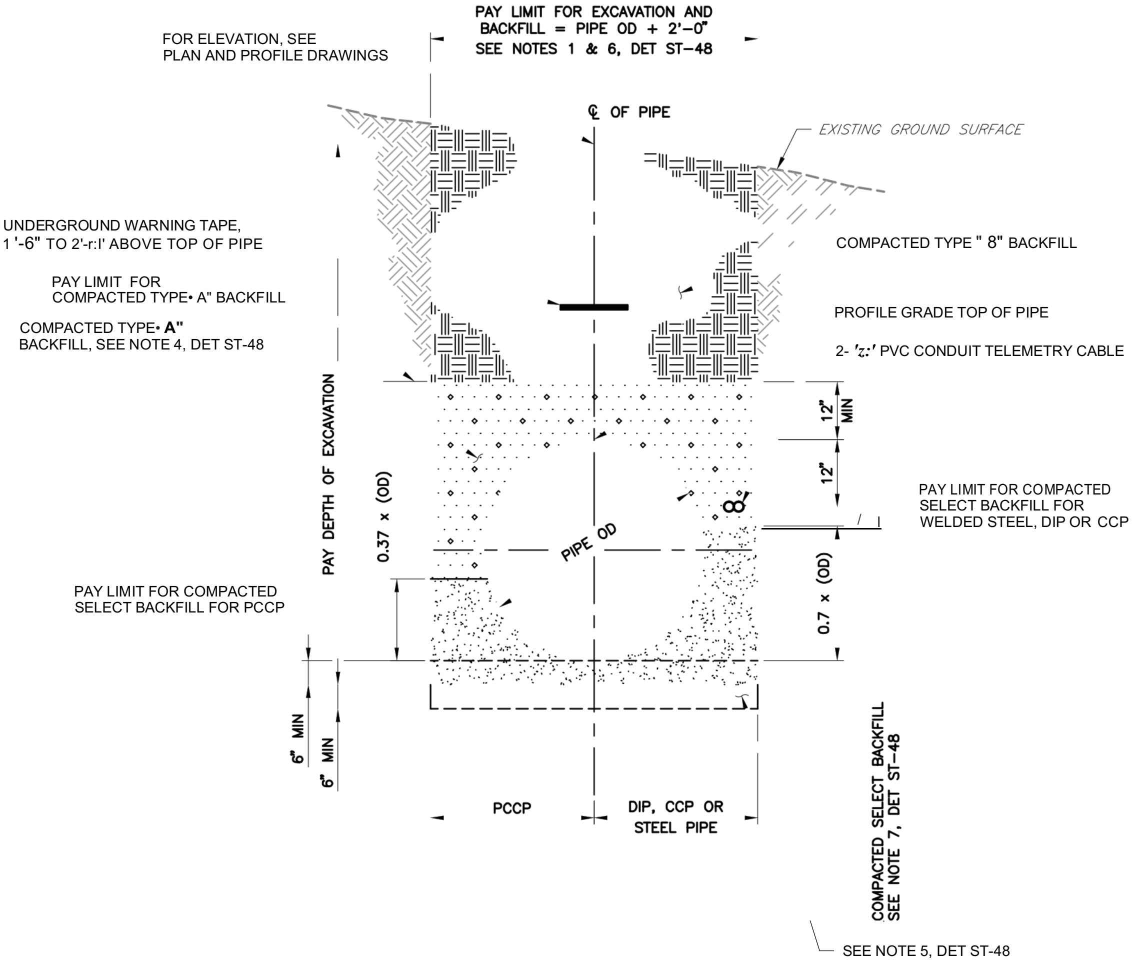
TYPICAL SECTION - TRENCH AT PAVED AREAS

SCALE

NOT TO SCALE

DETAIL NO.

ST-45


PAY LIMIT FOR EXCAVATION AND BACKFILL = PIPE OD + 2'-0" SEE NOTES 1 & 6, DET ST-48
FOR ELEVATION, SEE PLAN AND PROFILE DRAWINGS
℄ OF PIPE
EXISTING GROUND SURFACE
UNDERGROUND WARNING TAPE, 1 '-6" TO 2'-r:I' ABOVE TOP OF PIPE
PAY LIMIT FOR COMPACTED TYPE• A" BACKFILL
COMPACTED TYPE• A" BACKFILL, SEE NOTE 4, DET ST-48
COMPACTED TYPE " 8" BACKFILL
PROFILE GRADE TOP OF PIPE
2- 'z:' PVC CONDUIT TELEMETRY CABLE
PAY DEPTH OF EXCAVATION
0.37 x (OD)
12" MIN
12"
PAY LIMIT FOR COMPACTED SELECT BACKFILL FOR WELDED STEEL, DIP OR CCP
PIPE OD
0.7 x (OD)
PAY LIMIT FOR COMPACTED SELECT BACKFILL FOR PCCP
6" MIN
6" MIN
PCCP
DIP, CCP OR STEEL PIPE
COMPACTED SELECT BACKFILL SEE NOTE 7, DET ST-48
SEE NOTE 5, DET ST-48

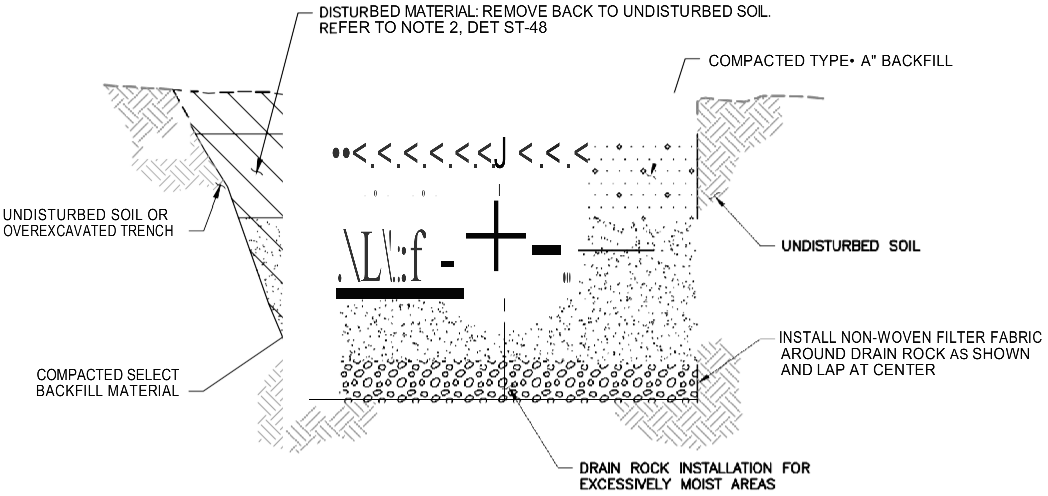
IYP-46 TYPICAL SECTION - TRENCH AT UNPAVED AREAS

SCALE: NTS

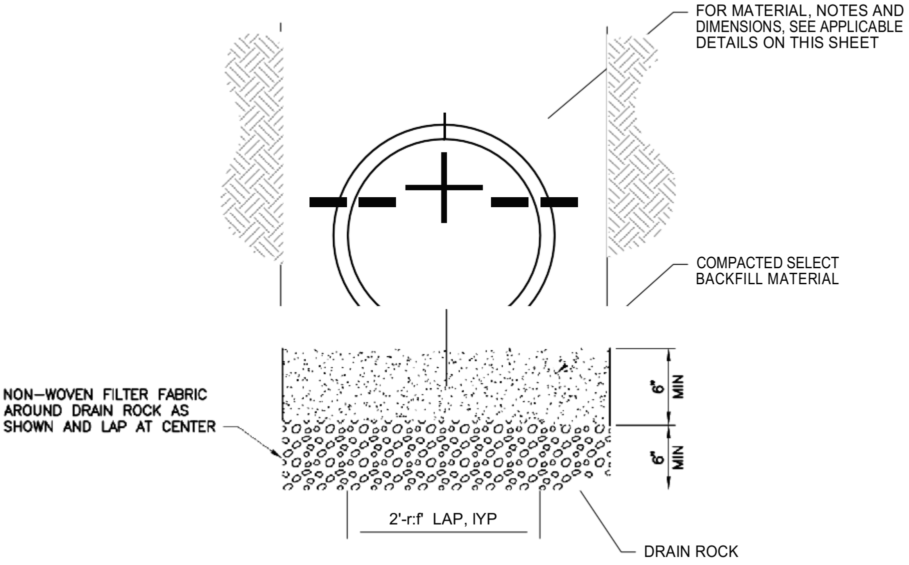
Plate No. 13B

Plans\Revised Details_2015-04\ST-46.dwg

DATE OCT 2014		PROJECT NAME AND SHEET DESCRIPTION: STANDARD DETAILS	SCALE NOT TO SCALE
DESIGN X.X.X			DETAIL NO. ST-46
DRAWN J. FORD		TYPICAL SECTION TRENCH AT UNPAVED AREAS	
RFY#			


DISTURBED MATERIAL: REMOVE BACK TO UNDISTURBED SOIL. REFER TO NOTE 2, DET ST-48
COMPACTED TYPE "A" BACKFILL
UNDISTURBED SOIL OR OVEREXCAVATED TRENCH
UNDISTURBED SOIL
INSTALL NON-WOVEN FILTER FABRIC AROUND DRAIN ROCK AS SHOWN AND LAP AT CENTER
COMPACTED SELECT BACKFILL MATERIAL
DRAIN ROCK INSTALLATION FOR EXCESSIVELY MOIST AREAS

OVEREXCAVATION OR CAVE-IN CORRECTIVE PROCEDURE


FOR MATERIAL, NOTES AND DIMENSIONS, SEE APPLICABLE DETAILS ON THIS SHEET
COMPACTED SELECT BACKFILL MATERIAL
NON-WOVEN FILTER FABRIC AROUND DRAIN ROCK AS SHOWN AND LAP AT CENTER
6" MIN
6" MIN
2'-0" LAP, TYP
DRAIN ROCK

DRAIN ROCK INSTALLATION

TRENCH EXCAVATION AND BACKFILL

SCALE: NTS

Plate No. 13C

DATE	OCT 2014
DESIGN	X.X.X
DRAWN	J. FORD
REV#	

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

STANDARD DETAILS

TRENCH EXCAVATION

AND BACKFILL

SCALE

NOT TO SCALE

DETAIL NO.

ST-47

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WIES;.

1. PAY LIMIT LINES INDICATE BOUNDARIES FOR CALCULATION OF QUANTmES OF PAVEMENT REMOVAL AND REPLACEMENT AND OF TRENCH EXCAVATION AND BACKFILL. TRENCHING SHALL CONFORM TO SAFETY REQUIREMENTS REGARDLESS OF PAY LIMITS SHOWN.
2. ALL OVEREXCAVATION OR DISTURBED NATIVE MATERIAL SHALL BE REMOVED FROM THE TRENCH AND BACKFILLED WITH THE SAME MATERIAL AS REQUIRED FOR TRENCH BACKFILL FOR THE DESIGNATED DEPTHS.
3. ALL EXISTING FACILITIES DAMAGED BY THE CONTRACTOR SHALL BE RESTORED IN ACCORDANCE WITH THE REQUIREMENTS OF THE AUTHORITY WITH JURISDICTION. REFER ALSO TO SPECIFICATIONS.
4. SELECT BACKFILL MATERIAL MAY BE USED AS AN ALTERNATIVE TO TYPE• A" AND TYPE "ff BACKFILL.
5. IF UNSUITABLE MATERIAL IS ENCOUNTERED, AS DEFINED IN THE SPECIFICATIONS, OVEREXCAVATE AND REPLACE WITH SUITABLE MATERIAL
6. . MINIMUM TRENCH WIDTH SHALL BE AS SHOWN. VARIATION OF THE TRENCH DIMENSIONS OR CONFIGURATION FROM THOSE SHOWN ON THE DRAWINGS MAY RESULT IN A CHANGE IN THE PIPE DESIGN. SEE SPECIFICATIONS.
7. FOR SOIL CEMENT BEDDING ALTERNATE, SEE DET ST-50.
8. EXISTING ROAD STRUCTURAL SECTIONS VARY. FOR ADDmONAL INFORMATION SEE SPECIFICATIONS.

ST-48
TYP

~~TRENCH EXCAVATION AND BACKFILL NOTES~~

SCALE: NTS

Plate No. 13D

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DATE	OCT 2014
DESIGN	X.X.X
DRAWN	J. FORD
REV#	

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PROJECT NAME AND SHEET DESCRIPTION:

STANDARD DETAILS

TRENCH EXCAVATION AND BACKFILL NOTES

SCALE
NOT TO SCALE

DETAIL NO.
ST-48

APPENDIX A

APPENDIX A
FIELD EXPLORATION

All the soil borings were drilled with drill rig using 4-inch diameter solid stem auger and 7-inch diameter hollow stem auger. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5 inches Inside Diameter (I. D.) Modified California Sampler or a 1.375 inches I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140 pounds hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings, Appendix A. (When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be converted to equivalent Standard Penetration Test sampler by multiplying a factor of 0.65). Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the boreholes were backfilled with cement grout.

The borings were drilled under the technical supervision of our engineers, who classified and continuously logged the soils encountered during drilling and supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were visually classified in the field according to the Unified Soil Classification System and then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the Log of Borings in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these logs of test borings depict subsurface soil conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface soil conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the subsurface soil conditions at these locations due to environmental changes.

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTS

C Consolidation (ASTM D 2435-04)
CL Collapse Potential (ASTM D 5333-03)
CP Compaction Curve (CTM 216 - 06)
CR Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
CU Consolidated Undrained Triaxial (ASTM D 4767-02)
DS Direct Shear (ASTM D 3080-04)
EI Expansion Index (ASTM D 4829-03)
M Moisture Content (ASTM D 2216-05)
OC Organic Content (ASTM D 2974-07)
P Permeability (CTM 220 - 05)
PA Particle Size Analysis (ASTM D 422-63[2002])
PI Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL Point Load Index (ASTM D 5731-05)
PM Pressure Meter
PP Pocket Penetrometer
R R-Value (CTM 301 - 00)
SE Sand Equivalent (CTM 217 - 99)
SG Specific Gravity (AASHTO T 100-06)
SL Shrinkage Limit (ASTM D 427-04)
SW Swell Potential (ASTM D 4546-03)
TV Pocket Torvane
UC Unconfined Compression - Soil (ASTM D 2166-06)
Unconfined Compression - Rock (ASTM D 2938-95)
UU Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW Unit Weight (ASTM D 4767-04)
VS Vane Shear (AASHTO T 223-96[2004])

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- Standard California Sampler
- Modified California Sampler
- Shelby Tube
- Piston Sampler
- NX Rock Core
- HQ Rock Core
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (short-term)
- Static Water Level Reading (long-term)

BORING RECORD LEGEND

PARIKH
Practicing in the Geosciences

MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015

Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate:
A-0A

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60} - Value (blows / foot)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Descriptor	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Descriptor	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE: This legend sheet provides descriptors and associated criteria for required soil description components only.

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010).

BORING RECORD LEGEND

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015

Job No.: 2014-101-T03

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Plate:
A-0B

LOGGED BY **L.S.Bhangoo**	BEGIN DATE **8-4-15**	COMPLETION DATE **8-4-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**	HOLE ID **B-01**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 64+10 "MD" LINE**	SURFACE ELEVATION **~383.5 ft**
DRILLING METHOD **Solid-Stem Auger**			DRILL RIG **CME 55**	BOREHOLE DIAMETER **4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC**			S˚PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Ground Water monitoring well installed**			GROUNDWATER READINGS DURING DRILLING / AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **21.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		Lean CLAY (CL); hard; brown; moist; low plasticity fines.												
	1				1	13 22 33	55	12	120	PP = >4.5	67				
381.50	2														
	3														
379.50	4		Lean CLAY with GRAVEL (CL); hard; brown; moist; low plasticity fines; (LL=28, PI=12).												
	5				2 & 3	12 18 27	45	12	126	PP = >4.5	89				CR PI
377.50	6														
	7														
375.50	8														
	9														
373.50	10				4 & 5	14 14 27	41	8			89				PA
	11		CLAYEY SAND with GRAVEL (SC); dense; brown; moist; (+#4=40%, -#200=20%).												
371.50	12														
	13														
369.50	14														
	15				6	22 25 42	67	6			67				
367.50	16		Very dense.												
	17														
365.50	18														
	19														
363.50	20				7	22 34 47	81	3			67				
	21														
361.50	22		Bottom of borehole at 21.5 ft bgs/Elev. 362.0 ft No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.												
	23														
359.50	24														
	25														

LOG OF TEST BORING

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-01	Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate: A-1

LOGGED BY **L.S.Bhangoo**	BEGIN DATE **8-4-15**	COMPLETION DATE **8-4-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**	HOLE ID **B-02**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 53+90 "MD" LINE**	SURFACE ELEVATION **~388.5 ft**
DRILLING METHOD **Solid-Stem Auger**			DRILL RIG **CME 55**	BOREHOLE DIAMETER **4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC**			S°PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with cement grout**			GROUNDWATER READINGS DURING DRILLING / AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **21.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		Lean CLAY with GRAVEL (CL); very stiff; brown; moist; fine GRAVEL; low plasticity fines; Coarse GRAVEL to 6" Cobbles.												
	1														
386.50	2				1	7 7 6	13	7	116	PP = 3.5	72				
	3														
384.50	4														
	5				2	7 13 16	29	8			78				
382.50	6														
	7														
380.50	8														
	9		CLAYEY SAND with GRAVEL (SC); dense; brown; moist; coarse to medium SAND; GRAVEL up to 2" dia.												
378.50	10				3	20 19 23	42	9			72				
	11														
376.50	12														
	13														
374.50	14														
	15				4	22 21 29	50	9			67				
372.50	16		Reddish brown; medium to fine SAND.												
	17														
370.50	18		Lean CLAY (CL); hard; brown; moist; medium plasticity fines.												
	19														
368.50	20				5	10 17 21	38	16	119	PP = 4.5 UU = 2.4	89				UU
	21														
366.50	22		Bottom of borehole at 21.5 ft bgs/Elev. 367.0 ft No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.												
	23														
364.50	24														
	25														

LOG OF TEST BORING

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-02	Job No.: 2014-101-T03

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Plate:
A-2

LOGGED BY **J. ZHANG**	BEGIN DATE **8-3-15**	COMPLETION DATE **8-3-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**	HOLE ID **B-03**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 43+75 "MD" LINE**	SURFACE ELEVATION **~393.5 ft**
DRILLING METHOD **Hollow-Stem Auger**			DRILL RIG **CME 55**	BOREHOLE DIAMETER **7 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC**			S*PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with cement grout**			GROUNDWATER READINGS DURING DRILLING AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **21.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		ASPHALT CONCRETE 3", AGGREGATE BASE 6".												
	1		Lean CLAY with SAND (CL); hard; brown; dry to moist; fine GRAVEL; coarse to fine SAND; low plasticity fines.												
391.50	2				1	13 28 30	58	6			83				
	3														
389.50	4														
	5				2	21 31 33	64	6			72				CR
387.50	6		Moist; trace GRAVEL; moderate cementation; Gravel size up to 2" in dia.												
	7														
385.50	8														
	9		Lean CLAY with GRAVEL (CL); hard; brown; moist; fine GRAVEL; medium to fine SAND; medium to high plasticity fines; (LL=34, PI=19).												
383.50	10				3	10 29 46	75	10			78				
	11														PI
381.50	12														
	13														
379.50	14														
	15				4	7 14 18	32	10			67				
377.50	16		Gravel size up to 2" in dia.												
	17														
375.50	18														
	19														
373.50	20				5	12 17 20	37	11			83				
	21														
371.50	22		Bottom of borehole at 21.5 ft bgs/Elev. 372.0 ft												
	23		No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.												
369.50	24														
	25														

LOG OF TEST BORING

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-03	Job No.: 2014-101-T03

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Plate:

A-3

LOGGED BY **J. ZHANG**	BEGIN DATE **8-3-15**	COMPLETION DATE **8-3-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**	HOLE ID **B-04**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 33+20 "MD" LINE**	SURFACE ELEVATION **~398.0 ft**
DRILLING METHOD **Hollow-Stem Auger**			DRILL RIG **CME 55**	BOREHOLE DIAMETER **7 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC, SPT**			S˚PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with cement grout**			GROUNDWATER DURING DRILLING READINGS / AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **21.2 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0	ASPHALT CONCRETE 3", AGGREGATE BASE 6".											
	1	SILT with SAND (ML); hard; brown; dry; fine SAND; (nonplastic fines).											
396.00	2		1	15 20 22	42	4	122	PP = >4.5	83				
	3												PI
394.00	4	Lean CLAY (CL); hard; brown; dry; low plasticity fines.											
	5		2	22 40 50/4"	90/10	5	129	PP = >4.5	81				
392.00	6												
	7												
390.00	8												
	9	CLAYEY GRAVEL with SAND (GC); dense; brown; dry to moist; low plasticity fines; (+#4=47%,-#200=13%).											
388.00	10		3	22 16 20	36	4			67				
	11												PA
386.00	12												
	13												
384.00	14	Lean CLAY with GRAVEL (CL); hard; gray; dry; low to medium plasticity fines; Gravel size up to 1" in dia.											
	15		4	22 39 46	85	6			67				
382.00	16												
	17												
380.00	18												
	19												
378.00	20	Brown; moist; coarse to medium SAND; low plasticity fines; Gravel size up to 1.5" in dia.	5	28 26 50/2"	76/8	29			86				
	21	Bottom of borehole at 21.2 ft bgs/Elev. 376.8 ft											
376.00	22	No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.											
	23												
374.00	24												
	25												

LOG OF TEST BORING

PARIKH
Practicing in the Geosciences

MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-04	Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

LOGGED BY **J. ZHANG**	BEGIN DATE **8-3-15**	COMPLETION DATE **8-3-15**	
BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**			HOLE ID **B-05**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**	BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 22+75 "MD" LINE**		SURFACE ELEVATION **~408.0 ft**
DRILLING METHOD **Hollow-Stem Auger**	DRILL RIG **CME 55**		BOREHOLE DIAMETER **7 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC**	S˚PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**		HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with cement grout**	GROUNDWATER READINGS DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **21.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		ASPHALT CONCRETE 3", AGGREGATE BASE 6".												
	1		Lean CLAY with SAND (CL); hard; brown; dry to moist.												
406.00	2				1	14 22 25	47				0				
	3														
404.00	4														
	5				2	14 22 25	47	13		PP = >4.5	89				CR
402.00	6		Moist; medium to fine SAND; low to medium plasticity fines.												
	7														
400.00	8														
	9														
398.00	10				3	11 12 18	30	9	116	PP = >4.5	83				PI
	11		Fine SAND; (LL=29, PI=15).												
396.00	12														
	13														
394.00	14														
	15				4	23 21 22	43	5		PP = >4.5	78				
392.00	16		Gray; medium to fine SAND; low plasticity fines.												
	17														
390.00	18														
	19														
388.00	20				5	25 38 43	81	6			78				
	21		Trace GRAVEL; Gravel size up to 1" in dia.												
386.00	22		Bottom of borehole at 21.5 ft bgs/Elev. 386.5 ft												
	23		No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.												
384.00	24														
	25														

LOG OF TEST BORING

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-05	Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

LOGGED BY **L.S.Bhangoo**	BEGIN DATE **8-4-15**	COMPLETION DATE **8-4-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**	HOLE ID **B-06**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 11+75 "MD" LINE**	SURFACE ELEVATION **~415.0 ft**
DRILLING METHOD **Solid-Stem Auger**			DRILL RIG **CME 55**	BOREHOLE DIAMETER **4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC, SPT**			S˚PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with cement grout**			GROUNDWATER READINGS DURING DRILLING AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **20.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		SILT with SAND (ML); hard; brown; moist; trace fine GRAVEL; (non-plastic fines).												
	1														
413.00	2				1	12 15 18	33	7	119	PP = >4.5	78				
	3														PI
411.00	4														
	5				2 & 3	13 9 8	17	8			72				
409.00	6		CLAYEY SAND with GRAVEL (SC); medium dense; brown; moist; Gravel size up to 1" in dia.												PA
	7														
407.00	8														
	9		Lean CLAY (CL); hard; brown; moist; low to medium plasticity fines.												
405.00	10		SILTY SAND with GRAVEL (SM); very dense; brown; moist; possible presence of cobbles.		4 & 5	50/3"	REF	5		PP = >4.5					
	11														
403.00	12														
	13														
401.00	14														
	15														
399.00	16														
	17														
397.00	18														
	19														
395.00	20				6	50/3"	REF				100				
	21		Bottom of borehole at 20.5 ft bgs/Elev. 394.5 ft No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.												
393.00	22														
	23														
391.00	24														
	25														

LOG OF TEST BORING

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-06	Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate: A-6

LOGGED BY **L.S.Bhangoo**	BEGIN DATE **8-4-15**	COMPLETION DATE **8-4-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**	HOLE ID **B-07**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 27+80 "MA" LINE**	SURFACE ELEVATION **~423.0 ft**
DRILLING METHOD **Hollow-Stem Auger**			DRILL RIG **CME 55**	BOREHOLE DIAMETER **7 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC**			S˚PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Ground Water monitoring well installed**			GROUNDWATER READINGS DURING DRILLING / AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **21.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0	ASPHALT CONCRETE 2", AGGREGATE BASE 3".												
	1	SILTY SAND with GRAVEL (SM); medium dense; brown; moist; fine SAND; weak cementation; Gravel size up to 1/4" india; weakly cement;.												
421.00	2			1	9 11 12	23	7			72				
	3													
419.00	4													
	5													
417.00	6	Very dense; coarse to fine SAND; weak cementation.		2&3	13 22 33	55	6			78				CR
	7													
415.00	8	SILTY SAND (SM); dense; brown; moist; fine GRAVEL; coarse to fine SAND.												
	9													
413.00	10	(+#4=10% , -#200=46%).		4&5	12 15 23	38	8			78				PA
	11													
411.00	12													
	13													
409.00	14													
	15			6	26 30 30	60	9			72				
407.00	16	Very dense.												
	17													
405.00	18	Poorly graded GRAVEL with SILT and SAND (GP-GM); very dense; brown; moist; medium SAND; Gravel size up to 2" in dia; (+#4=53% , -#200=11%); low to medium cementation.												
	19													
403.00	20			7	10 21 31	52	8			72				PA
	21													
401.00	22	Bottom of borehole at 21.5 ft bgs/Elev. 401.5 ft												
	23	No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.												
399.00	24													
	25													

LOG OF TEST BORING

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-07	Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate:

A-7

LOGGED BY **L.S.Bhangoo**	BEGIN DATE **8-5-15**	COMPLETION DATE **8-5-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**	HOLE ID **B-08**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 15+75 "MA" LINE**	SURFACE ELEVATION **~473.5 ft**
DRILLING METHOD **Solid-Stem Auger**			DRILL RIG **CME 55**	BOREHOLE DIAMETER **4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC, SPT**			S˚PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with cement grout**			GROUNDWATER READINGS DURING DRILLING AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **21.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		Lean CLAY with SAND (CL); very stiff; dark brown; moist; medium plasticity fines.												
	1														
471.50	2				1	4 8 11	19	21	107		72				
	3														
469.50	4														
	5				2	4 9 12	21				67				
467.50	6									UU = 1.7					UU
	7														
465.50	8														
	9														
463.50	10				3	12 15 33	48	17		PP = >4.5	67				
	11		Hard; (LL=42, PI=24).												PI
461.50	12														
	13														
459.50	14		CLAYEY SAND with GRAVEL (SC); very dense; brown; Gravel size up to 2" in dia; (+#4=43% , -#200=14%).												
	15				4	20 23 25	48	5			72				
457.50	16														PA
	17														
455.50	18		SANDY SILT (ML); hard; light brown; moist; trace fine GRAVEL.												
	19														
453.50	20				5	17 18 18	36	4			78				
	21														
451.50	22		Bottom of borehole at 21.5 ft bgs/Elev. 452.0 ft												
	23		No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.												
449.50	24														
	25														

LOG OF TEST BORING

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-08	Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate:
A-8

LOGGED BY **L.S.Bhangoo**	BEGIN DATE **8-5-15**	COMPLETION DATE **8-5-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**	HOLE ID **B-09**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 3+60 "MA" LINE**	SURFACE ELEVATION **~450.0 ft**
DRILLING METHOD **Hollow-Stem Auger**			DRILL RIG **CME 55**	BOREHOLE DIAMETER **7 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC, SPT**			S*PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with cement grout**			GROUNDWATER READINGS DURING DRILLING / AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **20.8 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		CLAYEY SAND (SC); dense; brown; moist; trace fine GRAVEL; low to medium plasticity fines; (+#4=3% , -#200=32%).												
	1														
448.00	2				1	15 28 44	72	11	108		67				PA
	3														
446.00	4		Lean CLAY (CL); very stiff; brown; moist; medium plasticity fines.												
	5				2 & 3	11 16 20	36	12 13			78				
444.00	6		CLAYEY SAND with GRAVEL (SC); very dense; brown; (+#4=24% , -#200=18%).												
	7				4			14			67				CR, PA
442.00	8		(No blow counts, corrosion sample).												
	9		Lean CLAY (CL); very dense; light gray; moist; Moderate to heavy cementation; (CLAYSTONE).												
440.00	10				5 & 6	13 48 50/5"	98/11	9 6			76				
	11														
438.00	12														
	13														
436.00	14														
	15				7	22 27 29	56	7			72				
434.00	16														
	17														
432.00	18														
	19														
430.00	20				8	43 50/4"	50/4	12			120				
	21		Bottom of borehole at 20.8 ft bgs/Elev. 429.2 ft												
428.00	22														
	23														
426.00	24														
	25														

No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.

LOG OF TEST BORING

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-09	Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate:
A-9

LOGGED BY **L.S.Bhangoo**	BEGIN DATE **8-5-15**	COMPLETION DATE **8-5-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**		HOLE ID **B-10**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 38+35 "MA" LINE**		SURFACE ELEVATION **~424.5 ft**
DRILLING METHOD **Hollow-Stem Auger**			DRILL RIG **CME 55**		BOREHOLE DIAMETER **7 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC, SPT**			S*PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**		HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with cement grout**			GROUNDWATER READINGS DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **21.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		ASPHALT CONCRETE 2", AGGREGATE BASE 3".												
			Fat CLAY (CH); very stiff; grayish brown; moist.												
	1														
422.50	2				1	3 5 7	12			PP = 2.8	89				
	3		(LL=54, PI=37).					21	101						PI
420.50	4														
	5				2	4 7 10	17				89				
418.50	6									UU = 1.3					UU
	7														
416.50	8														
			Lean CLAY (CL); hard; brown; moist; low plasticity fines.												
	9														
414.50	10				3	7 13 20	33			PP = >4.5	78				
	11							19	112						
412.50	12														
	13														
410.50	14		SILTY SAND with GRAVEL (SM); dense; brown; moist; fine GRAVEL; coarse to medium SAND; Gravel size up to 2" in dia.												
	15				4	18 21 25	46				67				
408.50	16							8							
	17														
406.50	18														
	19														
404.50	20				5	17 17 45	62				72				
	21		Very dense.					10							
			Bottom of borehole at 21.5 ft bgs/Elev. 403.0 ft												
402.50	22														
	23		No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.												
400.50	24														
	25														

LOG OF TEST BORING

PARIKH
Practicing in the Geosciences

MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-10	Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate:

A-10

LOGGED BY **J. ZHANG**	BEGIN DATE **8-3-15**	COMPLETION DATE **8-3-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**	HOLE ID **B-11**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 48+50 "MA" LINE**	SURFACE ELEVATION **~414.0 ft**
DRILLING METHOD **Hollow-Stem Auger**			DRILL RIG **CME 55**	BOREHOLE DIAMETER **7 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC, SPT**			S'PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with cement grout**			GROUNDWATER READINGS: DURING DRILLING / AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **21.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		ASPHALT CONCRETE 3", AGGREGATE BASE 6".												
	1		Lean CLAY with SAND (CL); very stiff to hard; brown; moist; fine SAND; low to medium plasticity fines.												
412.00	2				1	5 7 8	15			PP = 4.3 UU = 1.7	89				
	3														UU
410.00	4														
	5				2	7 12 18	30	21	112	PP = >4.5	89				
408.00	6		Hard; medium to fine SAND; medium to high plasticity fines; (LL=48, PI=27).												PI
	7														
406.00	8														
	9														
404.00	10				3	18 35 35	70	10		PP = >4.5	83				
	11		Trace fine GRAVEL; low plasticity fines.												CR
402.00	12														
	13														
400.00	14		SANDY lean CLAY with GRAVEL (CL); hard; brown; moist; fine GRAVEL; fine SAND; low plasticity fines.												
	15				4	22 50/5"	50/5	8	117		91				
398.00	16														
	17														
396.00	18														
	19														
394.00	20				5	38 43 42	85	6			78				
	21		Gravel size up to 1" in dia; Claystone observed at the bottom of sample.												
392.00	22		Bottom of borehole at 21.5 ft bgs/Elev. 392.5 ft No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.												
	23														
390.00	24														
	25														

PCI-CT 5 BR 2014-101-T03.GPJ TEMPLATE 7-22-11.GDT 2/3/16

LOG OF TEST BORING

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015 Boring ID: B-11 Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate:

A-11

LOGGED BY **L.S.Bhangoo**	BEGIN DATE **8-5-15**	COMPLETION DATE **8-5-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**	HOLE ID **B-12**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 64+95 "MA" LINE**	SURFACE ELEVATION **~397.5 ft**
DRILLING METHOD **Hollow-Stem Auger**			DRILL RIG **CME 55**	BOREHOLE DIAMETER **7 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC, SPT**			S˚PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with cement grout**			GROUNDWATER READINGS DURING DRILLING / AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **21.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		Lean CLAY with SAND (CL); hard; brown; dry to moist; trace GRAVEL; Gravel size up to 0.25" in dia.												
	1														
395.50	2				1	12 9 12	21	7	97	PP = >4.5	72				
	3														
393.50	4		SILTY SAND with GRAVEL (SM); dense; brown; moist; trace GRAVEL; fine SAND; Gravel size up to 0.25" in dia; (+#4=19%, -#200=22%).												
	5				2	15 17 20	37	5	88		72				
391.50	6														PA
	7														
389.50	8														
	9														
387.50	10				3	16 18 29	47	7			56				
	11		Some dark gray sand; possibly some cobbles.												
385.50	12														
	13														
383.50	14		CLAYEY SAND with GRAVEL (SC); dense; brown; moist; fine SAND; Gravel size up to 0.25" in dia.												
	15				4	10 13 20	33	9			67				
381.50	16														
	17														
379.50	18														
	19														
377.50	20				5	19 20 21	41	9			72				
	21		Very dense; fine GRAVEL; medium to fine SAND.												
375.50	22		Bottom of borehole at 21.5 ft bgs/Elev. 376.0 ft No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.												
	23														
373.50	24														
	25														

LOG OF TEST BORING

PARIKH
Practicing in the Geosciences

MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-12	Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate:
A-12

LOGGED BY **J. ZHANG**	BEGIN DATE **8-3-15**	COMPLETION DATE **8-3-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**	HOLE ID **B-13**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 69+75 "MA" LINE**	SURFACE ELEVATION **~394.5 ft**
DRILLING METHOD **Hollow-Stem Auger**			DRILL RIG **CME 55**	BOREHOLE DIAMETER **7 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC, SPT**			S˚PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Backfilled with cement grout**			GROUNDWATER READINGS DURING DRILLING AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **21.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		ASPHALT CONCRETE 3", AGGREGATE BASE 7".												
	1		Lean CLAY with SAND (CL); hard; brown; dry; fine SAND; low plasticity fines.												
392.50	2				1	22 14 20	34	7	115	PP = >4.5	83				
	3														
390.50	4		GRAVELLY lean CLAY (CL); hard; brown; dry; low plasticity fines; Gravel size up to 1.5" in dia.												
	5				2	29 45 50/4"	95/10	6		PP = >4.5	81				
388.50	6														CR
	7														
386.50	8														
	9		Lean CLAY with SAND (CL); hard; brown; dry; fine SAND; low to medium plasticity fines.												
384.50	10				3	24 25 23	48	6	103	PP = >4.0	78				
	11														
382.50	12														
	13														
380.50	14														
	15				4	37 50/3"	50/3	7	95	PP = >4.0	100				
378.50	16														
	17														
376.50	18		CLAYEY GRAVEL with SAND (GC); dense; brown; moist; low plasticity fines; Gravel size up to 1.5" in dia; (+#4=51%, -#200=18%).												
	19														
374.50	20				5	27 20 14	34	8			78				
	21														PA
372.50	22		Bottom of borehole at 21.5 ft bgs/Elev. 373.0 ft No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.												
	23														
370.50	24														
	25														

LOG OF TEST BORING

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-13	Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate:
A-13

LOGGED BY **L.S.Bhangoo**	BEGIN DATE **8-5-15**	COMPLETION DATE **8-5-15**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **See Boring Location Map**	HOLE ID **B-14**
DRILLING CONTRACTOR **Geo-Ex Subsurface Exploration**			BOREHOLE LOCATION (Offset, Station, Line) **0.00' Rt Sta 75+10 "MA" LINE**	SURFACE ELEVATION **~395.0 ft**
DRILLING METHOD **Hollow-Stem Auger**			DRILL RIG **CME 55**	BOREHOLE DIAMETER **7 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **MC, SPT**			S*PT HAMMER TYPE **140 lbs Auto Hammer with 30" Drop**	HAMMER EFFICIENCY, ERi **84%**
BOREHOLE BACKFILL AND COMPLETION **Ground Water monitoring well installed**			GROUNDWATER READINGS DURING DRILLING AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **21.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. PP in Comp.Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		Lean CLAY with SAND (CL); hard; brown; moist; trace GRAVEL; fine SAND; medium plasticity fines; Gravel size up to 1.5" in dia.												
	1														
393.00	2				1	8 16 25	41	8	116	PP = >4.5	67				
	3														
391.00	4														
	5				2	21 26 25	51	7	134	PP = >4.5	72				
389.00	6		(LL=27, PI=10).												PI
	7														
387.00	8		SILTY SAND with GRAVEL (SM); very dense; brown; moist; coarse to medium SAND; Gravel size up to 1.5" in dia.												
	9														
385.00	10				3	18 25 47	72	12	120		67				
	11		(+#4=39%, -#200=16%).												PA
383.00	12														
	13														
381.00	14														
	15				4	11 12 20	32	5			56				
379.00	16		Dense.												
	17														
377.00	18		CLAYEY GRAVEL with SAND (GC); very dense; brown; fine GRAVEL; coarse to medium SAND.												
	19														
375.00	20				5	13 17 24	41	11	125		67				
	21														
373.00	22		Bottom of borehole at 21.5 ft bgs/Elev. 373.5 ft												
	23		No groundwater was encountered during drilling. The groundwater level is anticipated to vary with time due to seasonal fluctuations, variations in yearly rainfall, water elevations in nearby creeks, surface and subsurface flows, and environmental factors that may not be present at the time of the investigation.												
371.00	24														
	25														

LOG OF TEST BORING

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT

SANTA CLARA COUNTY, CALIFORNIA

Date: 8/3/2015	Boring ID: B-14	Job No.: 2014-101-T03

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate:
A-14

APPENDIX B

APPENDIX B
LABORATORY TESTS

Classification Tests

The field classification of the samples was visually verified in the laboratory according to the Unified Soil Classification System. The results are presented on "Log of Test Borings", Appendix A.

Moisture-Density

The natural moisture contents were determined for selected undisturbed samples of the soils in general accordance with ASTM D-2216 and dry unit weights were calculated based on natural moisture contents and total unit weights. This information was used to classify and correlate the soils. The results are presented on Plates B-1A and B-1B "Summary of Laboratory Test Results", Appendix B.

Atterberg Limits

The Atterberg Limits were determined for selected samples of the fine-grained materials. These results were used to classify the soils, as well as to obtain an indication of the expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with ASTM D-4318. The results of the test are presented on Plate B-2, "Plasticity Chart", Appendix B.

Grain Size Classification

Grain size classification tests (ASTM D-422) were performed on selected samples of granular soil to aid in the classification. The results are presented on Plates B-3A through B-3C, "Grain Size Distribution Curves", Appendix B.

Unconsolidated-Undrained Triaxial Tests

Unconsolidated-undrained triaxial tests were performed on selected undisturbed samples in accordance with ASTM D 2850. The results are presented on Plate B-4, "Unconsolidated-Undrained Triaxial Test", Appendix B.

Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistivity tests were performed according to California Test Method CT-643. Sulfate (California Test Method CT-417) and chloride (California Test Method CT-422) tests were performed by Sunland Analytical. The test results are presented on Plates B-5A through B-5G, Appendix B.

APPENDIX B
LABORATORY TESTS

R-Value Tests

R-value tests were performed on selected bulk samples. The tests were performed according to ASTM D 3284. The test results are presented on Plates B-6A through B-6G, Appendix B.

Modified Proctor Tests

Modified Proctor tests were performed on selected composite samples to find their maximum dry density and optimum moisture content. The tests were performed according to ASTM D 1557. The test results are presented on Plates B-7A through B-7D, Appendix B.

Borehole	Sample Number	Depth	Classi-fication	Water Content	Dry Density	Liquid Limit	Plastic Limit	Plasticity Index	% > Sieve 4	% < Sieve 200	Shear Strength (tsf)
B-01	1	2.0	CL	12.2	120.0						
B-01	2	5.0	CL	-	-						
B-01	3	5.5	CL	11.8	126.5	28	16	12			
B-01	4	10.0	SC	-	-						
B-01	5	10.5	SC	7.9	-				39.5	20.4	
B-01	6	16.0	SC	6.3	-						
B-01	7	21.0	SC	3.4	-						
B-02	1	3.0	CL	7.4	116.2						
B-02	2	6.0	CL	8.1	-						
B-02	3	11.0	SC	9.3	-						
B-02	4	16.0	SC	8.6	-						
B-02	5	21.0	CL	15.6	118.5						UU = 2.4
B-03	1	3.0	CL	6.1	-						
B-03	2B	5.5	CL	-	-						
B-03	2	6.0	CL	5.8	-						
B-03	3	11.0	CL	9.5	-	34	15	19			
B-03	4	16.0	CL	10.3	-						
B-03	5	21.0	CL	11.4	-						
B-04	1	3.0	ML	3.7	121.8	NP	NP	NP			
B-04	2	6.0	CL	4.6	129.1						
B-04	3	11.0	GC	4.0	-				46.8	13.0	
B-04	4	16.0	CL	6.3	-						
B-04	5	21.0	CL	28.6	-						
B-05	2B	5.5	CL	-	-						
B-05	2	6.0	CL	13.0	-						
B-05	3	11.0	CL	9.2	116.5	29	15	14			
B-05	4	16.0	CL	5.2	-						
B-05	5	21.0	CL	5.6	-						
B-06	1	3.0	ML	7.4	118.8	NP	NP	NP			
B-06	2	5.0	ML	-	-						
B-06	3	5.5	ML	8.2	-				28.8	14.0	
B-06	4	9.5	CL	4.9	-						
B-06	5	10.5	CL	-	-						
B-07	1	3.0	SM	7.3	-						
B-07	2	6.0	SM	-	-						
B-07	3	6.5	SM	5.9	-						
B-07	4	10.0	SM	-	-						
B-07	5	10.5	SM	7.5	-				9.7	45.8	
B-07	6	16.0	SM	9.3	-						
B-07	7	21.0	GP-GM	7.9	-				52.8	10.5	
B-08	1	3.0	CL	20.7	107.4						
B-08	2	6.0	CL	-	-						UU = 1.7
B-08	3	11.0	CL	17.2	-	42	19	23			

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT
SANTA CLARA COUNTY, CALIFORNIA

JOB NO: 2014-101-T03

PLATE NO: B-1A

Borehole	Sample Number	Depth	Classi-fication	Water Content	Dry Density	Liquid Limit	Plastic Limit	Plasticity Index	% > Sieve 4	% < Sieve 200	Shear Strength (tsf)
B-08	4	16.0	SC	4.6	-				42.4	13.6	
B-08	5	21.0	ML	3.6	-						
B-09	1	3.0	SC	11.1	107.5				3.2	31.6	
B-09	2	5.0	CL	12.0	-						
B-09	3	5.5	CL	13.1	-						
B-09	4	7.0	SC	13.7	-				24.0	17.5	
B-09	5	10.0	CL	9.2	-						
B-09	6	10.5	CL	6.4	-						
B-09	7	16.0	CL	6.7	-						
B-09	8	20.0	CL	12.0	-						
B-10	1	3.0	CH	21.4	101.4	54	17	37			
B-10	2	6.0	CH	-	-						UU = 1.3
B-10	3	11.0	CL	18.7	111.5						
B-10	4	16.0	SM	7.7	-						
B-10	5	21.0	SM	10.4	-						
B-11	1	3.0	CL	-	-						UU = 1.7
B-11	2	6.0	CL	20.5	112.4	48	21	27			
B-11	3	11.0	CL	9.7	-						
B-11	4	15.5	CL	8.2	117.3						
B-11	5	21.0	CL	5.5	-						
B-12	1	3.0	CL	6.5	96.8						
B-12	2	6.0	SM	5.2	88.4				19.1	21.8	
B-12	3	11.0	SM	6.7	-						
B-12	4	16.0	SC	8.9	-						
B-12	5	21.0	SC	9.3	-						
B-13	1	3.0	CL	6.6	115.4						
B-13	2	6.0	CL	6.1	-						
B-13	3	11.0	CL	5.6	103.4						
B-13	4	15.0	CL	6.9	94.8						
B-13	5	21.0	GC	7.9	-				51.2	17.6	
B-14	1	3.0	CL	7.7	115.9						
B-14	2	6.0	CL	7.4	133.8	27	17	10			
B-14	3	11.0	SM	11.5	120.4				39.0	16.4	
B-14	4	16.0	SM	4.7	-						
B-14	5	21.0	GC	10.6	124.8						

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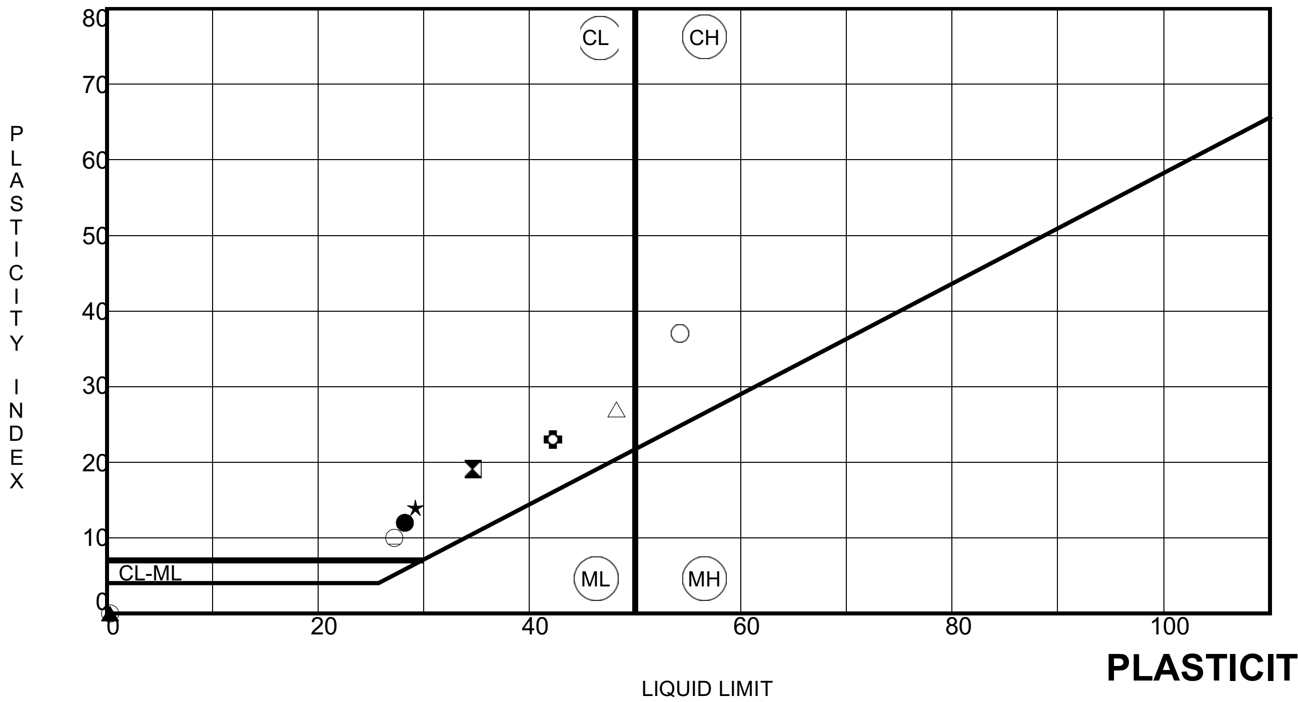
MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT
SANTA CLARA COUNTY, CALIFORNIA

JOB NO: 2014-101-T03

PLATE NO: B-1B



PLASTICITY

	BOREHOLE	SAMPLE #	DEPTH	LL	PL	PI	Fines	Classification
●	**B-01**	**3**	**5.5**	**28**	**16**	**12**		**Lean CLAY with GRAVEL**
⧗	**B-03**	**3**	**11.0**	**34**	**15**	**19**		**Lean CLAY with GRAVEL**
▲	**B-04**	**1**	**3.0**	**NP**	**NP**	**NP**		**SILT with SAND**
★	**B-05**	**3**	**11.0**	**29**	**15**	**14**		**Lean CLAY with SAND**
⊙	**B-06**	**1**	**3.0**	**NP**	**NP**	**NP**		**SILT with SAND**
✿	**B-08**	**3**	**11.0**	**42**	**19**	**23**		**Lean CLAY with SAND**
○	**B-10**	**1**	**3.0**	**54**	**17**	**37**		**Fat CLAY**
△	**B-11**	**2**	**6.0**	**48**	**21**	**27**		**Lean CLAY with SAND**
⊗	**B-14**	**2**	**6.0**	**27**	**17**	**10**		**Lean CLAY with SAND**

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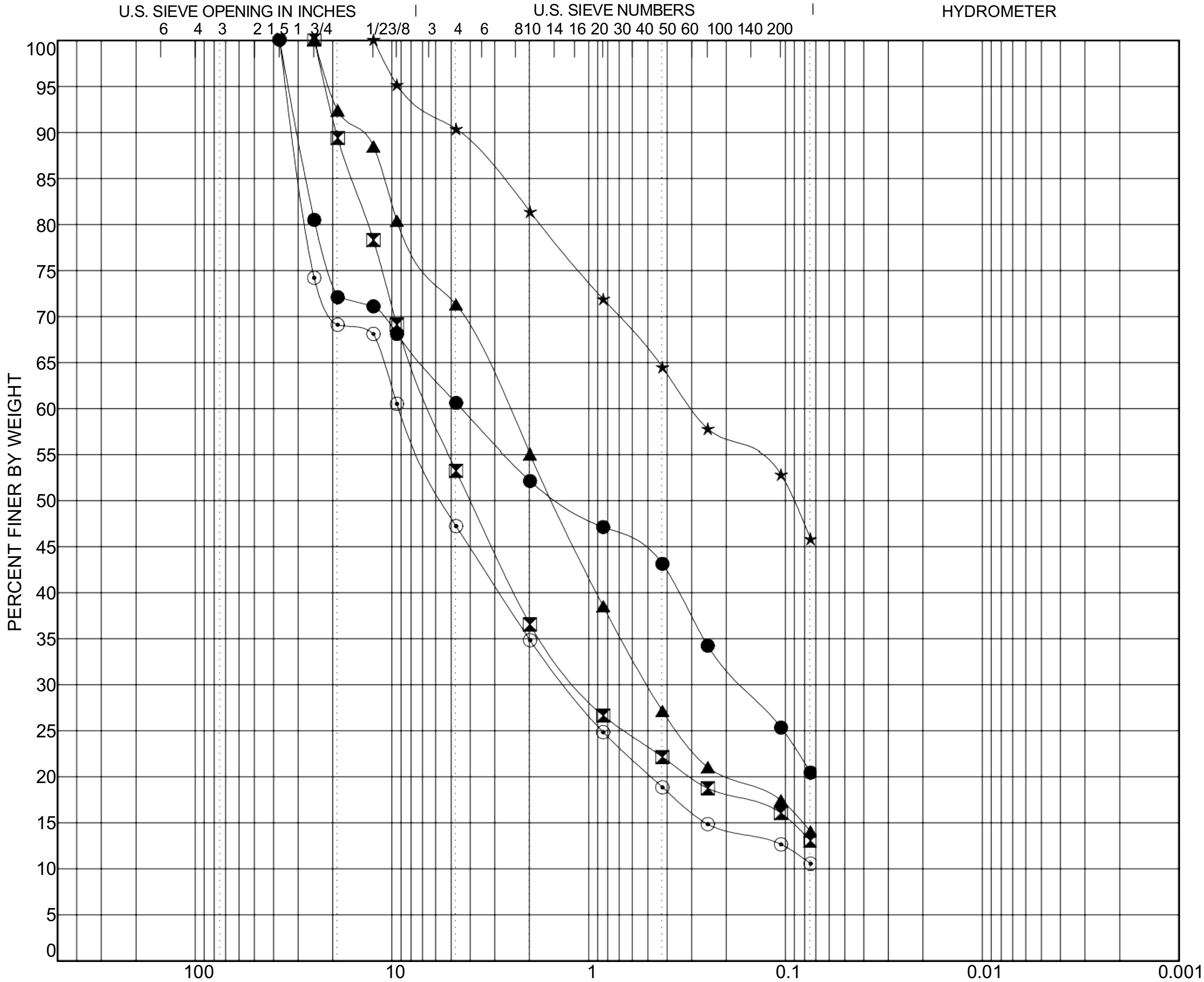
MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT
SANTA CLARA COUNTY, CALIFORNIA

JOB NO: 2014-101-T03

PLATE NO: B-2



GRAIN SIZE DISTRIBUTION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING	SAMPLE #	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-01**	**5**	**10.5**	**CLAYEY SAND with GRAVEL**					
⧗	**B-04**	**3**	**11.0**	**CLAYEY GRAVEL with SAND**					
▲	**B-06**	**3**	**5.5**	**SILT with SAND**					
★	**B-07**	**5**	**10.5**	**SILTY SAND**					
⊙	**B-07**	**7**	**21.0**	**Poorly graded GRAVEL with SILT and SAND**				**2.74**	**134.68**

	BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-01**	**5**	**10.5**	**37.5**	**4.512**	**0.167**		**39.5**	**40.1**	**20.4**	
⧗	**B-04**	**3**	**11.0**	**25**	**6.401**	**1.14**		**46.8**	**40.2**	**13.0**	
▲	**B-06**	**3**	**5.5**	**25**	**2.612**	**0.507**		**28.8**	**57.2**	**14.0**	
★	**B-07**	**5**	**10.5**	**12.5**	**0.3**			**9.7**	**44.5**	**45.8**	
⊙	**B-07**	**7**	**21.0**	**37.5**	**9.303**	**1.326**		**52.8**	**36.7**	**10.5**	

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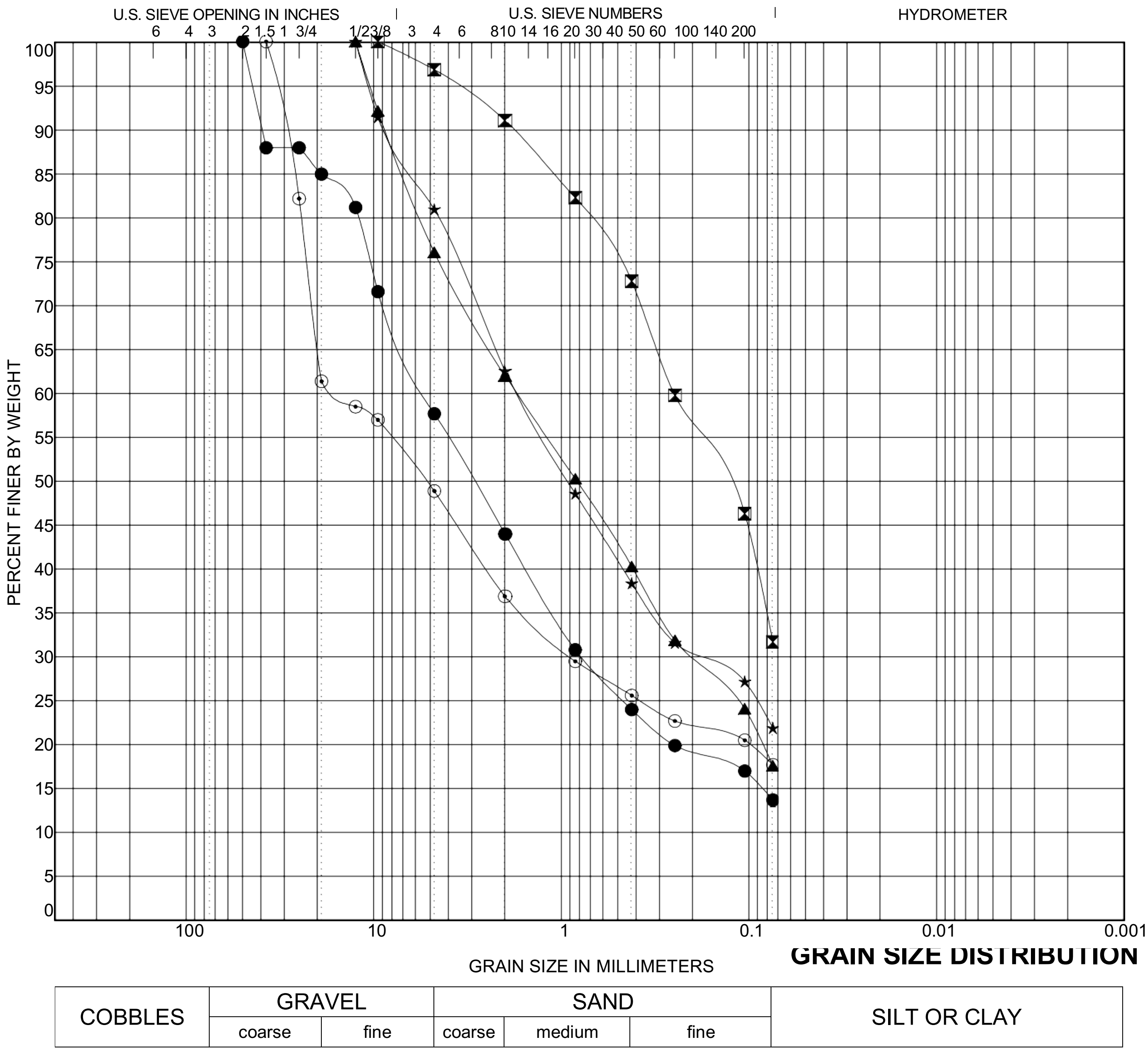
MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT
SANTA CLARA COUNTY, CALIFORNIA

JOB NO: 2014-101-T03
PLATE NO: B-3A



GRAIN SIZE DISTRIBUTION

	BORING	SAMPLE #	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-08**	**4**	**16.0**	**CLAYEY SAND with GRAVEL**					
⧗	**B-09**	**1**	**3.0**	**CLAYEY SAND**					
▲	**B-09**	**4**	**7.0**	**CLAYEY SAND with GRAVEL**					
★	**B-12**	**2**	**6.0**	**SILTY SAND with GRAVEL**					
⊙	**B-13**	**5**	**21.0**	**CLAYEY GRAVEL with SAND**					

	BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-08**	**4**	**16.0**	**50**	**5.354**	**0.791**		**42.4**	**44.0**	**13.6**	
⧗	**B-09**	**1**	**3.0**	**9.5**	**0.253**			**3.2**	**65.2**	**31.6**	
▲	**B-09**	**4**	**7.0**	**12.5**	**1.741**	**0.205**		**24.0**	**58.5**	**17.5**	
★	**B-12**	**2**	**6.0**	**12.5**	**1.717**	**0.187**		**19.1**	**59.1**	**21.8**	
⊙	**B-13**	**5**	**21.0**	**37.5**	**15.748**	**0.911**		**51.2**	**31.2**	**17.6**	

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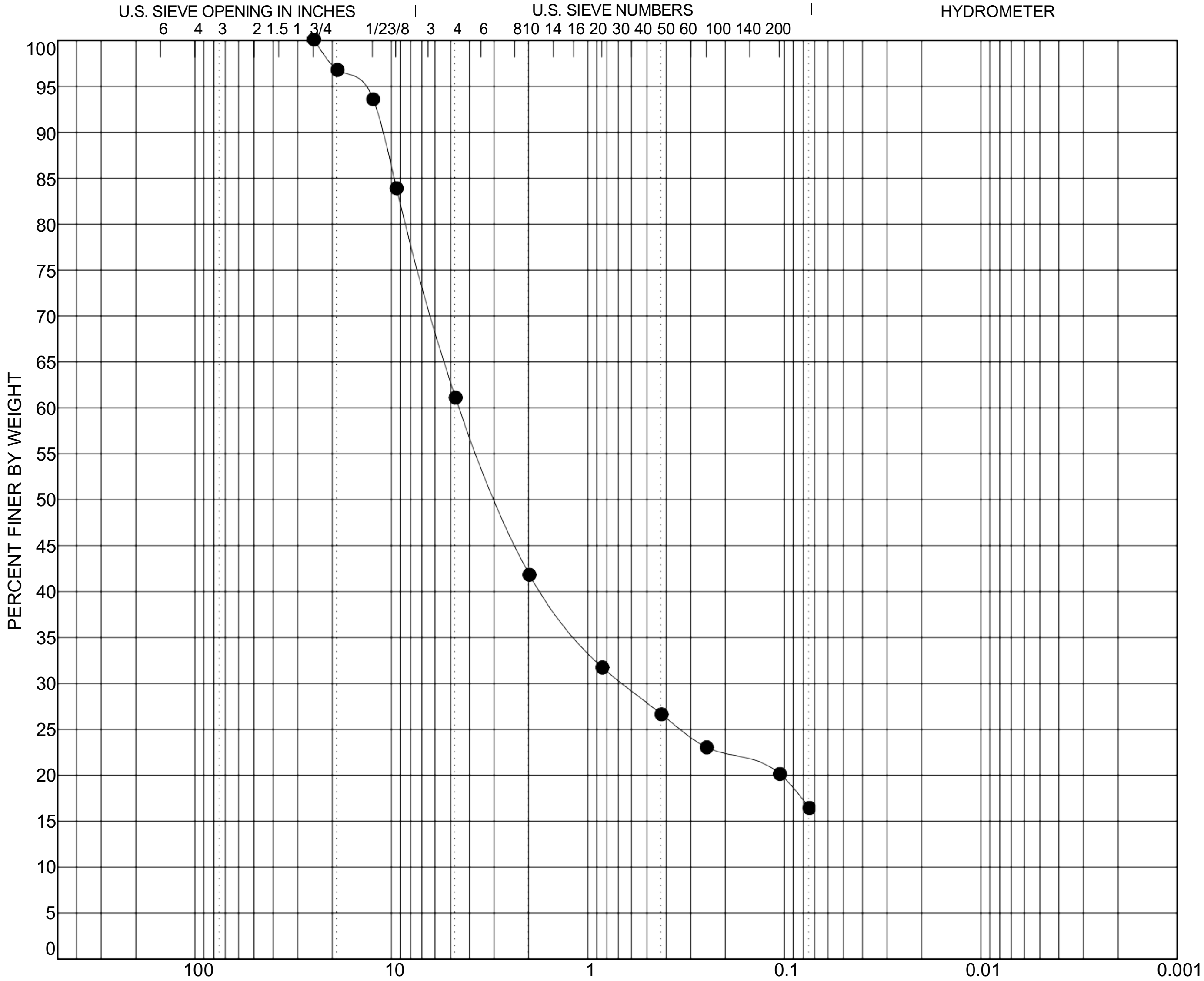
MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT
SANTA CLARA COUNTY, CALIFORNIA

JOB NO: 2014-101-T03

PLATE NO: B-3B



GRAIN SIZE DISTRIBUTION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING	SAMPLE #	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-14**	**3**	**11.0**	**SILTY SAND with GRAVEL**					

	BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-14**	**3**	**11.0**	**25**	**4.541**	**0.675**		**39.0**	**44.6**	**16.4**	

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MAIN AVE. AND MADRONE PIPELINES RESTORATION PROJECT
SANTA CLARA COUNTY, CALIFORNIA

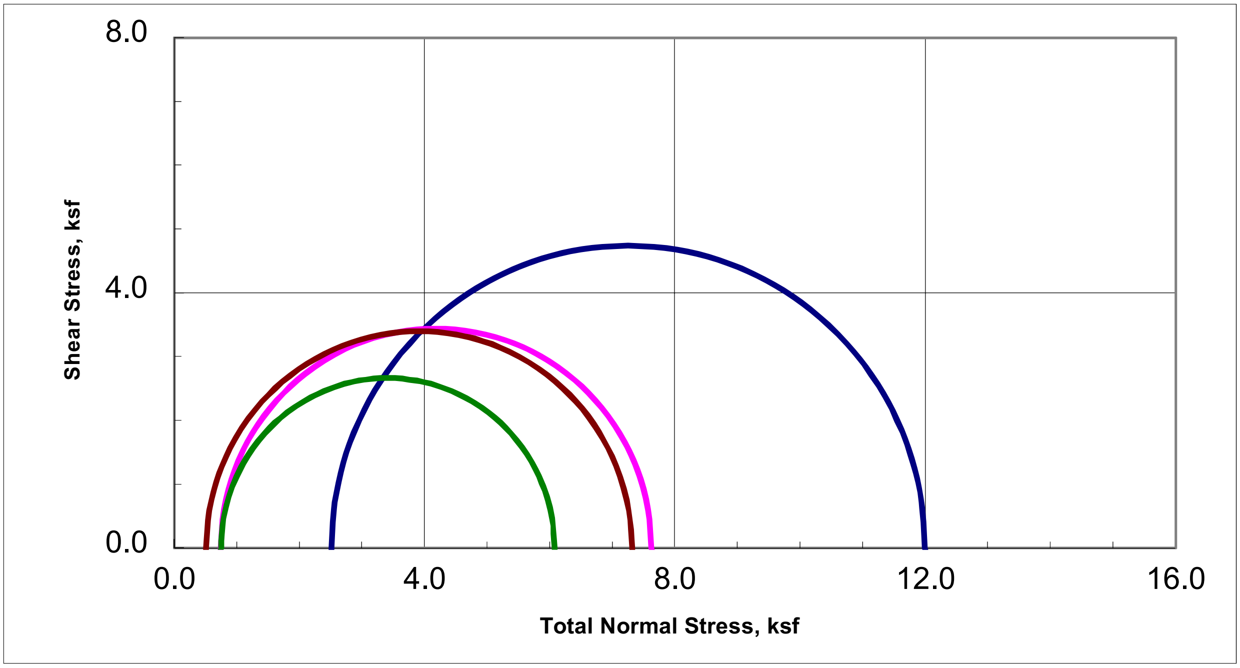
JOB NO: 2014-101-T03

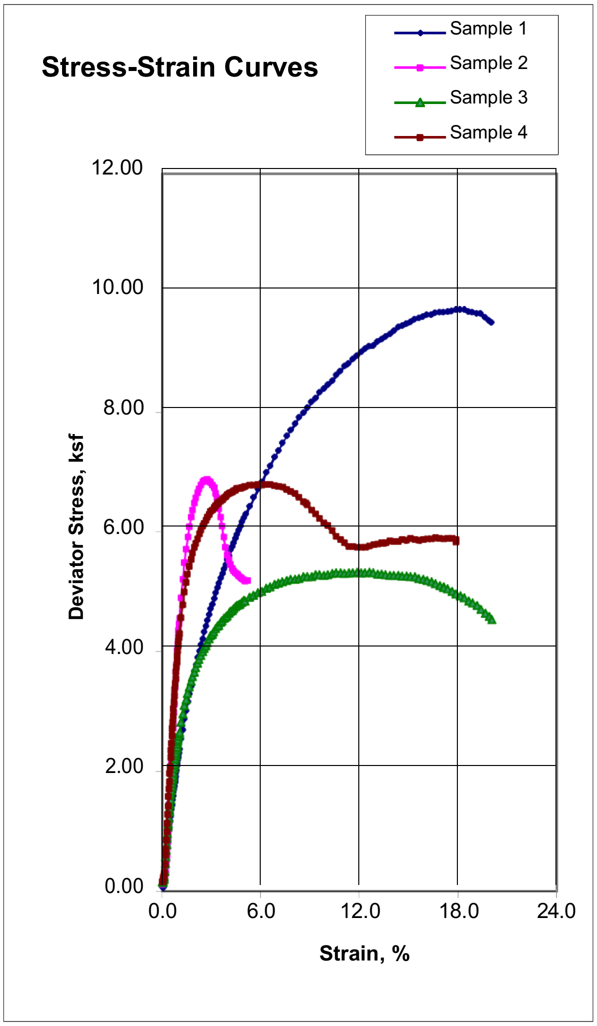
PLATE NO: B-3C

Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

Unconsolidated-Undrained Triaxial Test

ASTM D2850




Stress-Strain Curves

Sample Data

	1	2	3	4
Moisture %	15.6	21.4	23.9	21.7
Dry Den,pcf	118.5	107.2	101.2	106.0
Void Ratio	0.423	0.601	0.665	0.590
Saturation %	99.7	97.7	97.0	99.1
Height in	5.01	5.00	4.99	5.02
Diameter in	2.40	2.41	2.41	2.41
Cell psi	17.4	5.1	5.2	3.5
Strain %	15.00	2.75	11.84	6.56
Deviator, ksf	9.487	6.887	5.332	6.806
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.050	0.050	0.050	0.050
Job No.:	**157-328**			
Client:	**Parikh Consultants**			
Project:	**2014-101-T03**			
Boring:	B-2	B-8	B-10	B-11
Sample:	#5	#2	#2	#1
Depth ft:	21	6	6	3

Visual Soil Description

Sample #	
1	Yellowish Brown Sandy CLAY
2	Brown Sandy CLAY
3	Very Dark Gray CLAY
4	Dark Gray CLAY, trace Gravel

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/19/2015
Date Submitted 08/13/2015

To: Nasir Ahmad
Parikh Consultants, Inc.
2360 Qume Dr. Suite A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horneyr;t:.µ
General Manager \ Lab Manager {

The reported analysis was requested for the following location:
Location 2014-10i-T03 Site ID: B-1-2@ 5 FT.
Thank you for your business.

* For future reference to this analysis please use SUN# 70228-146404.

EVALUATION FOR SOIL CORROSION

Soil pH	6.27	
Minimum Resistivity	2.01 ohm-cm (xl000)	
Chloride	23.6 ppm	00.00236 %
Sulfate	33.8 ppm	00.00338 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CJ.',, DOT Test #417, Chloride CA IiOT Test #422

PLATE NO.: B-5A

Sunland Analytical
11419 Sunrise Gold Circle, 1110
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/19/2015
Date Submitted 08/13/2015

To: Nasir Ahmad
Parikh Consultants, Inc.
2360 Qume Dr. Suite A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horney
Generc!,l Manager \ Lab Manager

The reported analysis was requested for the following location:
Location 2014-101-T03 Site ID: B-3-2B@ 5.5 FT.
Thank you for your business.

* For future reference to this analysis please use SUN# 70228-146405.

EVALUATION FOR SOIL CORROSION

Soil pH	7.08		
Minimum Resistivity	3.62 ohm-cm (x1000)		
Chloride	14.8 ppm	00.00148	%
Sulfate	23.3 ppm	00.00233	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test .#4.17, Chloride :CA DOT T!;;l,St #4:;!2

PLATE NO.: B-5B

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/19/2015
Date Submitted 08/13/2015

To: Nasir Ahmad
Parikh Consultants, Inc.
2360 Qume Dr. Suite A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horney/{,p
General Manager \ Lab Manager/

The reported analysis was requested for the following location:
Location 2014-101-T03 Site ID: B-5-2B@ 5.5 FT.
Thank you for your business.

* For future reference to this analysis please use SUN# 70228-146406.

EVALUATION FOR SOIL CORROSION

Soil pH	7,87		
Minimum Resistivity	4.29	ohm-cm (x1000)	
Chloride	13.6 ppm	00.00136	%
Sulfate	26.0 ppm	00.00260	%

METHODS
pH and Min.Resistivity CA DOT Test #643
SulfatE;i CA DOT Test #417,· Chloride CA DOT Test #422

PLATE NO.: B-5C

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/19/2015
Date Submitted 08/13/2015

To: Nasir Ahmad
Parikh Consultants, Inc.
2360 Qume Dr. Suite A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horney,t',}4
General Manager \ Lab Manager \

The reported analysis was requested for the following location:
Location 2014-101-T03 Site ID: B-7-2@ 6 FT.
Thank you for your business.

* For future reference to this analysis please use SUN# 70228-146407.

EVALUATION FOR SOIL CORROSION

Soil pH	7.03	
Minimum Resistivity	1.61 ohm-cm (x1000)	
Chloride	12.3 ppm	00.00123 %
Sulfate	7,8 ppm	00.00078 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417,· Chloride CA DOT Test #422

PLATE NO.: B-5D

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/19/2015
Date Submitted 08/13/2015

To: Nasir Ahmad
Parikh Consultants, Inc.
2360 Qume Dr. Suite A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horney,-f,,
General Manager \ Lab Manager 1

The reported analysis was requested for the following location:
Location 2014-101-T03 Site ID: B-9-4@ 7 FT.
Thank you for your business.

* For future reference to this analysis please use SUN# 70228-146408.

EVALUATION FOR SOIL CORROSION

Soil pH	7.66		
Minimum Resistivity	0.80	ohm-cm (x1000)	
Chloride	6.5 ppm	00.00065	%
Sulfate	11.8 ppm	00.00118	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sul:Ea.te. CA DOT Test #417, Chlo.ride CA DOT Test #422

PLATE NO.: B-5E

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/19/2015
Date Submitted 08/13/2015

To: Nasir Ahmad
Parikh Consultants, Inc.
2360 Qume Dr. Suite A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horney4A
General Manager \ Lab Manager\

The reported analysis was requested for the following location:
Location 2014-101-T03 Site ID: B-11-3@ 11 FT.
Thank you for your business.

* For future reference to this analysis please use SUN# 70228-146409.

EVALUATION FOR SOIL CORROSION

Soil pH	7.56		
Minimum Resistivity	1.02	ohm-cm (x1000)	
Chloride	232.9 ppm	00.02329	%
Sulfate	13.2 ppm	00.00132	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417,. Chloride CA DOT Te.st #422

PLATE NO.: B-5F

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/19/2015
Date Submitted 08/13/2015

To: Nasir Ahmad
Parikh Consultants, Inc.
2360 Qume Dr. Suite A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager l

The reported analysis was requested for the following location:
Location 2014-101-T03 Site ID: B-13-2@ 6 FT.
Thank you for your business.

* For future reference to this analysis please use SUN# 70228-146410.

EVALUATION FOR SOIL CORROSION

Soil pH	6.94	
Minimum Resistivity	5.23 ohm-cm (xlOOO)	
Chloride	11.9 ppm	00.00119 %
Sulfate	13.9 ppm	00.00139 %

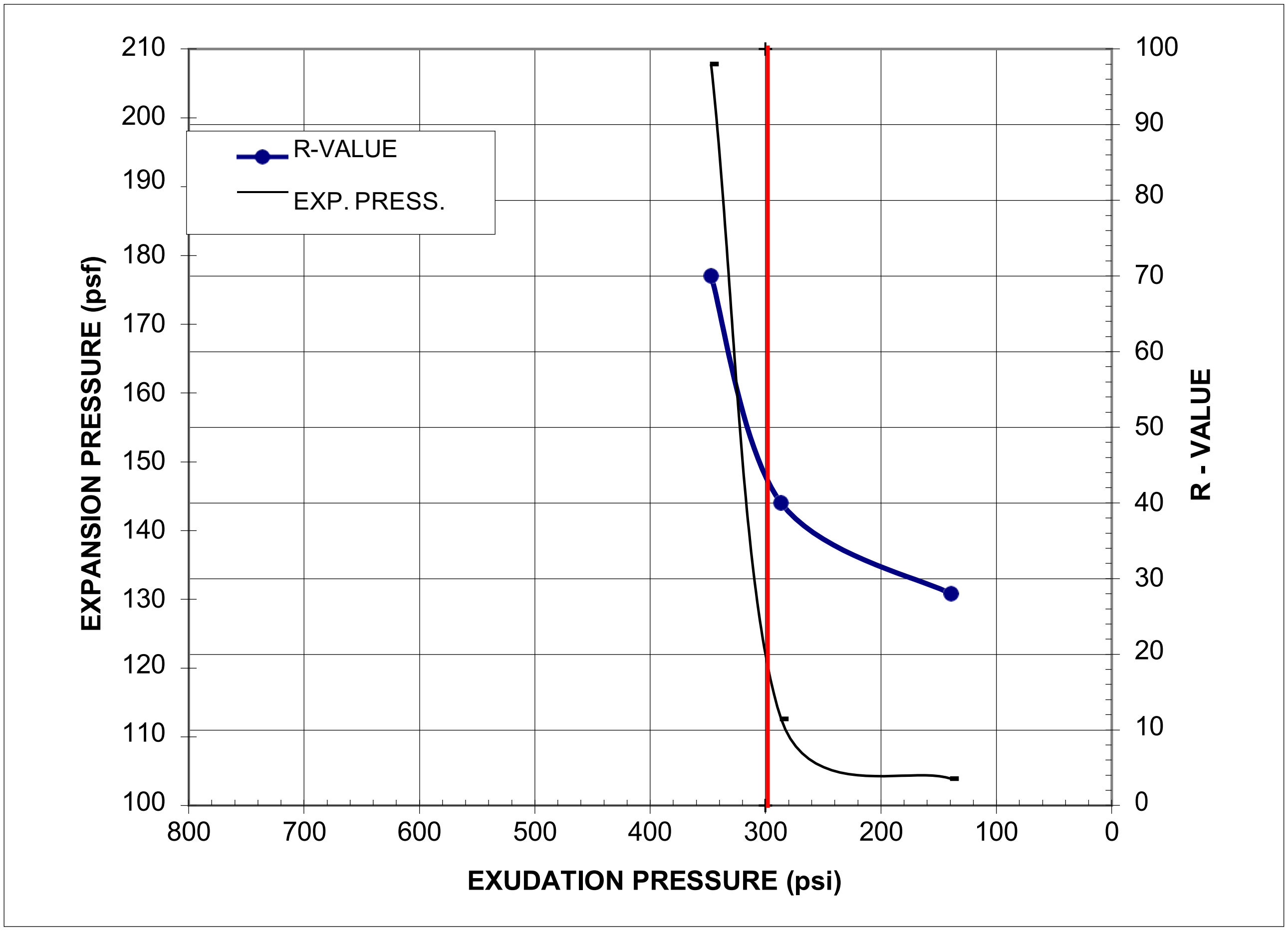
METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate ·cA DO'l' Test #417, Chloride CA DOT Test,#422

PLATE NO.: B-5G

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	Main Ave and Madrone Pipelines Restoration Project			Date:	9/3/15
Client:	SCVWD			Project #:	2014-101-T03
Sample #:	B-2	Depth:	0'-5'	Lab #:	M 949
Location	Morgan Hill, CA	Onsite / Native		Sample Date:	
Material :	Sandy Clay			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	139.2	286.6	347.3
Expansion Pressure, psf	103.9	112.6	207.8
R-Value	28	40	70
Moisture Content at Test, %	12.0	11.1	10.3
Dry Density at Test, pcf	121.7	124.5	124.6

R-Value @ 300 psi Exudation Pressure = 43

Expansion Pressure @300 psi Exudation, psf = **125**

Minimum R-Value Requirement:

Comments:

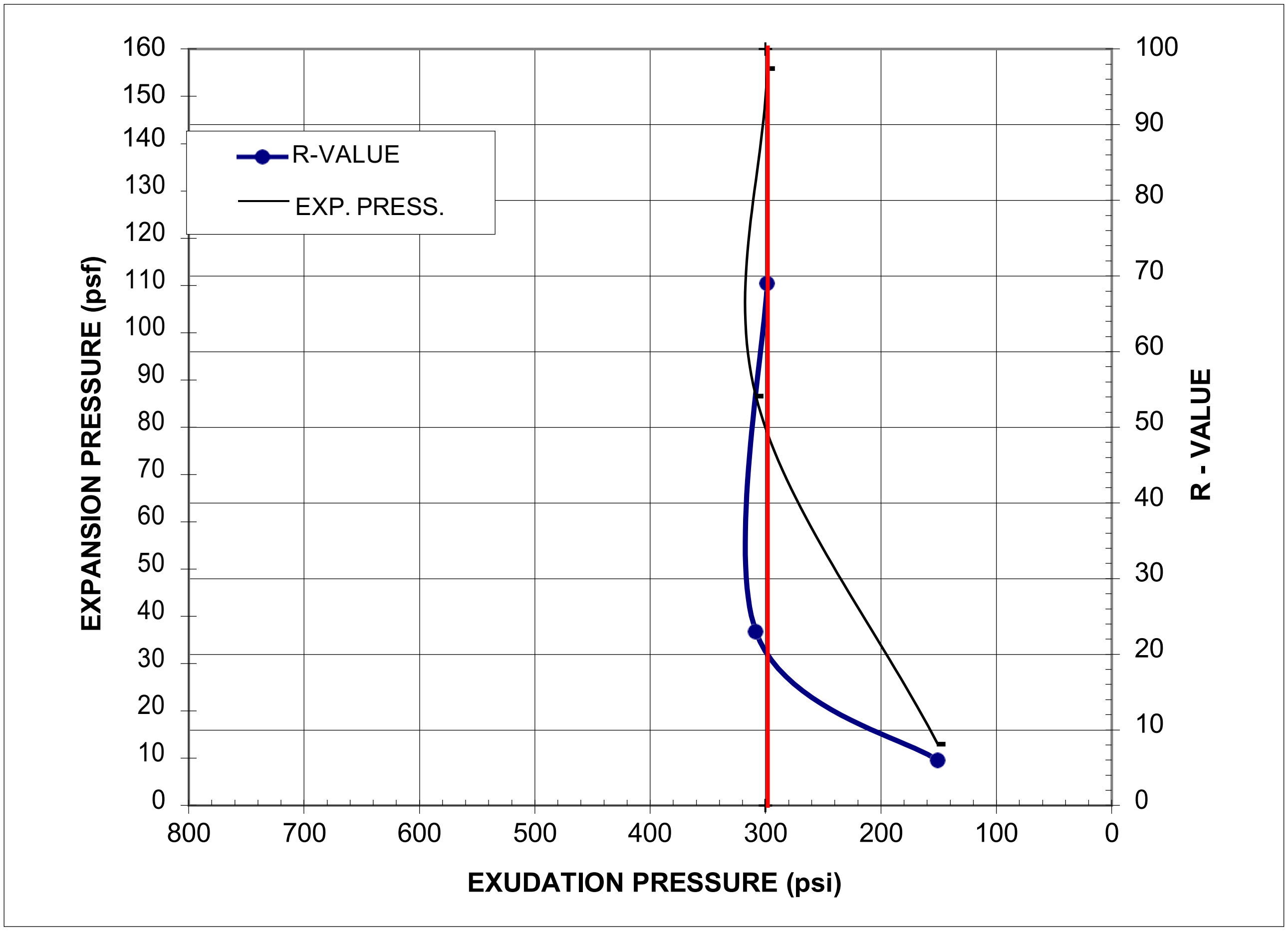
Report By: Nasir Ahmad

PLATE NO.: B-6A

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	Main Ave and Madrone Pipelines Restoration Project	Date:	10/14/15
Client:	SCVWD	Project #:	2014-101-T03
Sample #:	B-4 Depth: 0'-5'	Lab #:	M 949
Location	Morgan Hill, CA Onsite / Native	Sample Date:	
Material :	Sandy Silt	Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	150.9	308.5	298.67
Expansion Pressure, psf	12.99	86.6	155.88
R-Value	6	23	69
Moisture Content at Test, %	10.6	8.9	8.0
Dry Density at Test, pcf	121.0	127.2	128.7

R-Value @ 300 psi Exudation Pressure =	**20**	Expansion Pressure @300 psi Exudation, psf =	**80**
Minimum R-Value Requirement:			

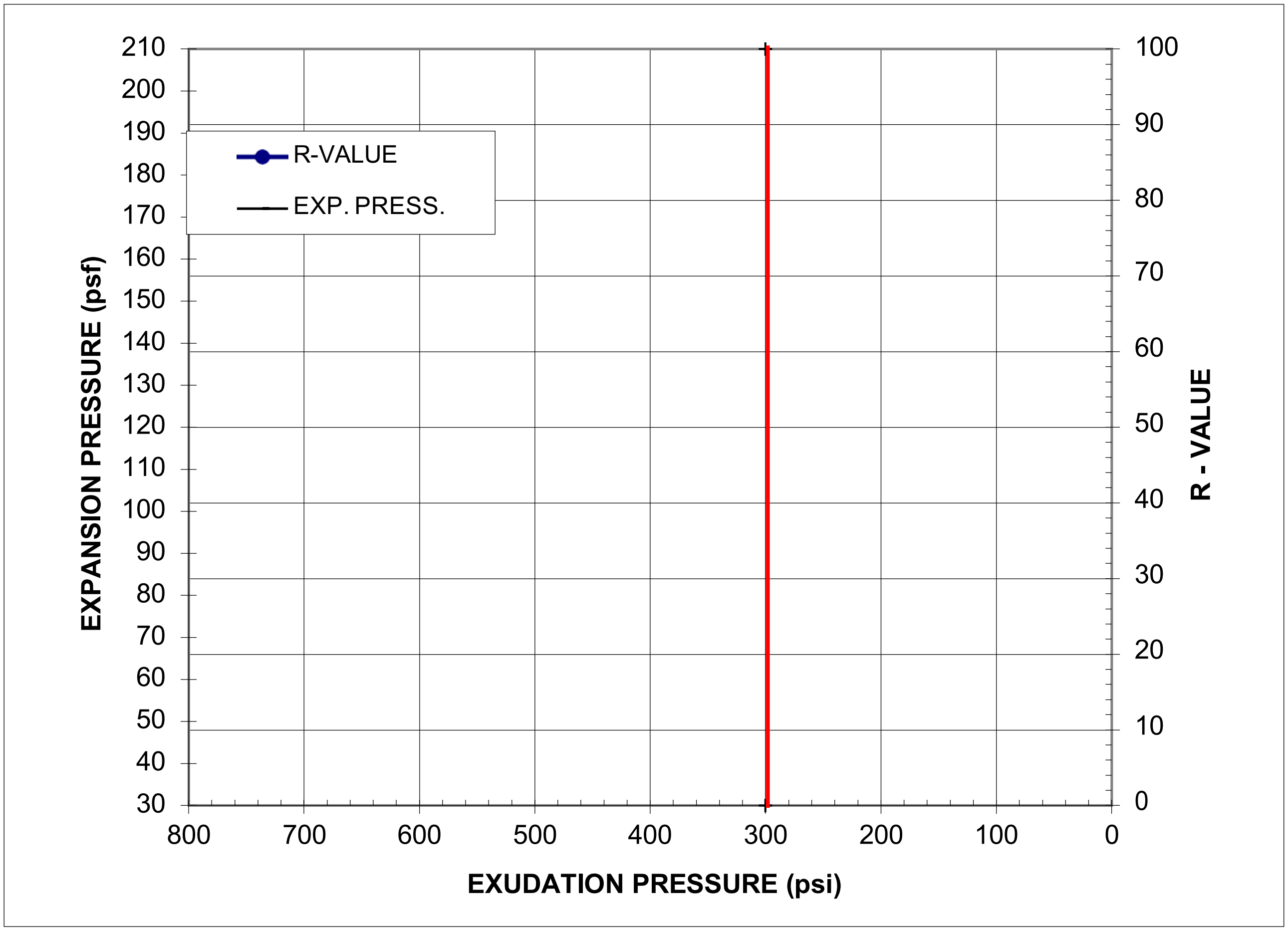
Comments:

Report By: Nasir Ahmad	PLATE NO.: B-6B

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	Main Ave and Madrone Pipelines Restoration Project	Date:	10/8/15
Client:	SCVWD	Project #:	2014-101-T03
Sample #:	B-5 Depth: 0'-5'	Lab #:	M 949
Location	Morgan Hill, CA Onsite / Native	Sample Date:	
Material :	Brown Expansive Clay	Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi			
Expansion Pressure, psf			
R-Value		0	0
Moisture Content at Test, %			
Dry Density at Test, pcf			

R-Value @ 300 psi Exudation Pressure = less then 5 Expansion Pressure @300 psi Exudation, psf =

Minimum R-Value Requirement:

Comments:

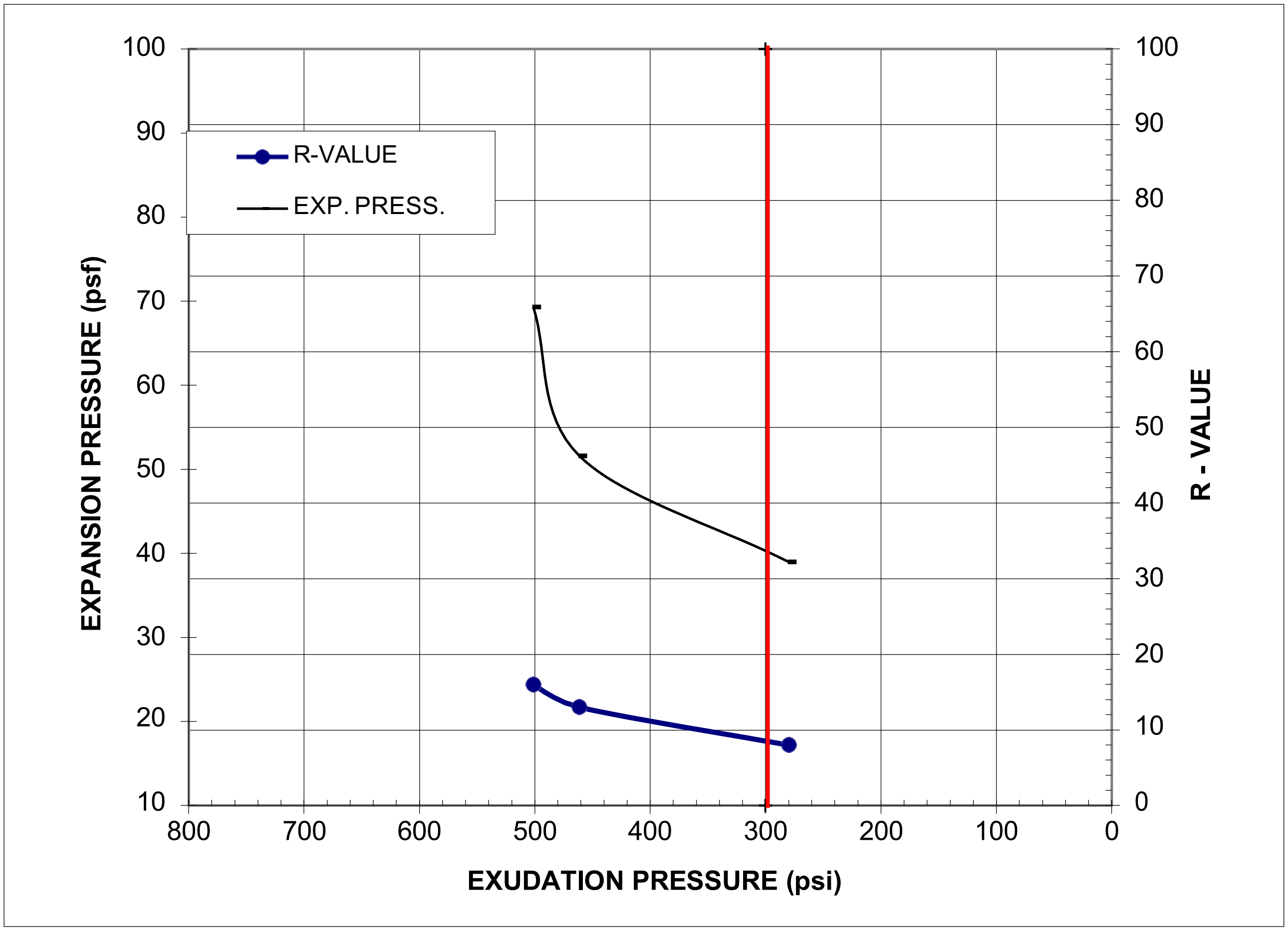
Report By: Nasir Ahmad

PLATE NO.: B-6C

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	Main Ave and Madrone Pipelines Restoration Project			Date:	10/26/15
Client:	SCVWD			Project #:	2014-101-T03
Sample #:	B-10	Depth:	0'-5'	Lab #:	M 949
Location	Morgan Hill, CA	Onsite / Native		Sample Date:	
Material :	Dark Brown Fat Clay+ Gravel			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	279.9	461.48	501.3
Expansion Pressure, psf	38.97	51.6	69.3
R-Value	8	13	16
Moisture Content at Test, %	20.2	18.3	17.4
Dry Density at Test, pcf	113.2	107.9	108.5

R-Value @ 300 psi Exudation Pressure = 9

Expansion Pressure @300 psi Exudation, psf = **40**

Minimum R-Value Requirement:

Comments:

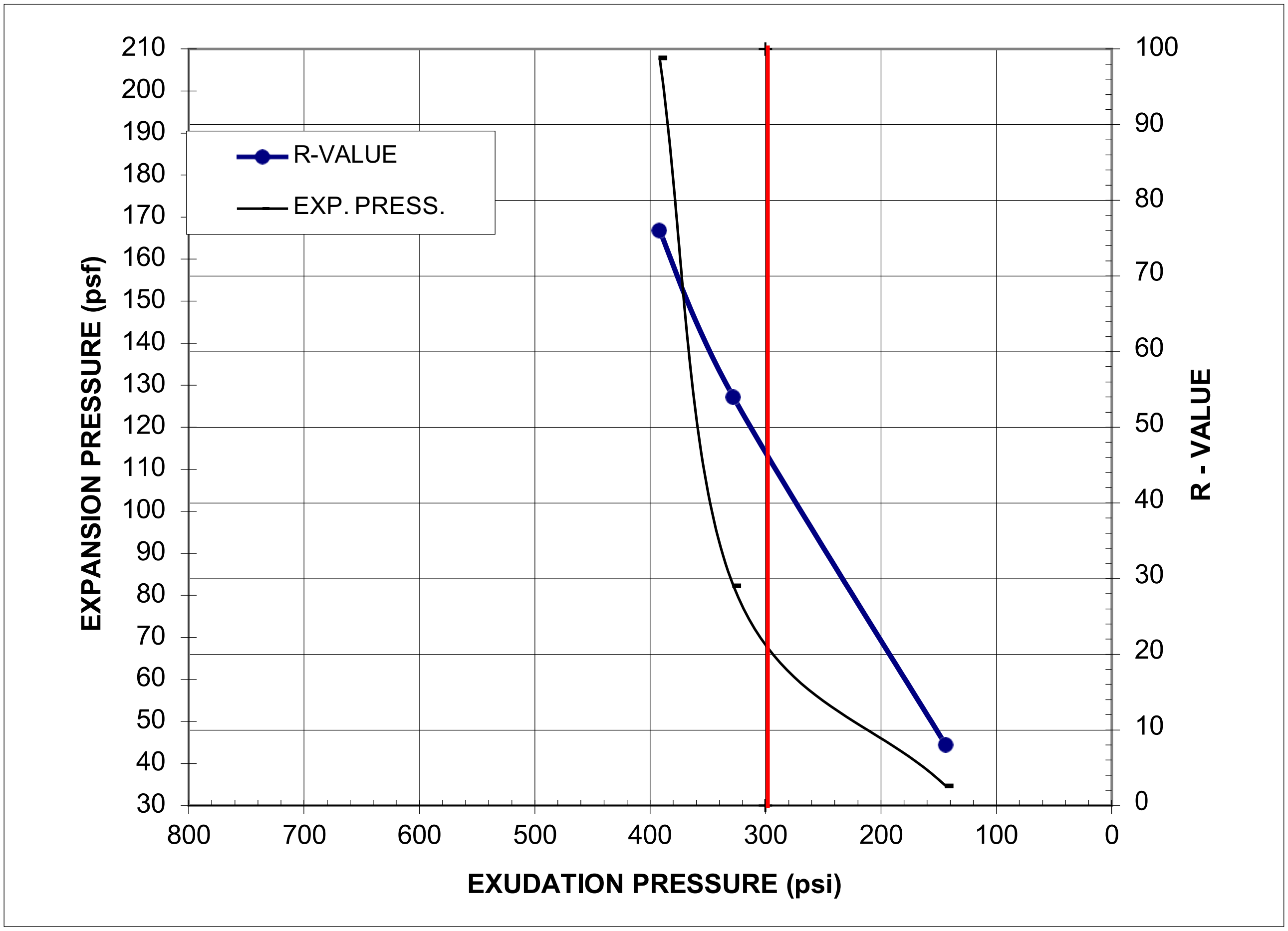
Report By: Nasir Ahmad

PLATE NO.: B-6D

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	Main Ave and Madrone Pipelines Restoration Project			Date:	10/8/15
Client:	SCVWD			Project #:	2014-101-T03
Sample #:	B-11	Depth:	0'-5'	Lab #:	M 949
Location	Morgan Hill, CA	Onsite / Native		Sample Date:	
Material :	Sandy Silt			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	143.8	327.9	391.8
Expansion Pressure, psf	34.64	82.27	207.84
R-Value	8	54	76
Moisture Content at Test, %	11.3	9.5	8.6
Dry Density at Test, pcf	122.8	128.9	128.2

R-Value @ 300 psi Exudation Pressure = 47

Expansion Pressure @300 psi Exudation, psf = **68**

Minimum R-Value Requirement:

Comments:

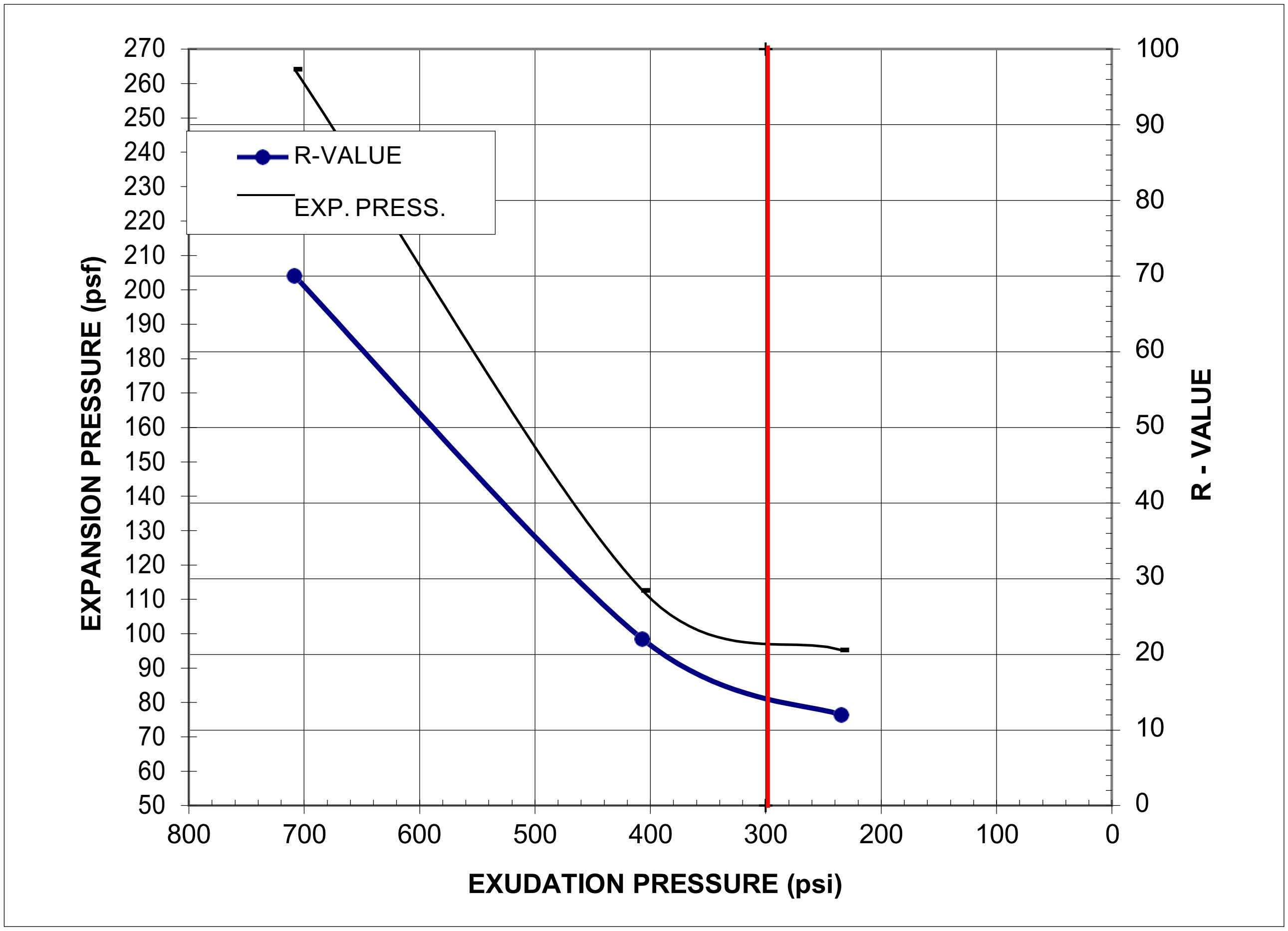
Report By: Nasir Ahmad

PLATE NO.: B-6E

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	Main Ave and Madrone Pipelines Restoration Project	Date:	2/4/16
Client:	SCVWD	Project #:	2014-101-T03
Sample #:	B-7 Depth: 0'-5'	Lab #:	M 949
Location	Morgan Hill, CA Onsite / Native	Sample Date:	
Material :	Silty Sand	Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	234.5	406.8	708.25
Expansion Pressure, psf	95.26	112.58	264.13
R-Value	12	22	70
Moisture Content at Test, %	12.8	11.9	11.0
Dry Density at Test, pcf	119.2	121.7	123.7

R-Value @ 300 psi Exudation Pressure = 14

Expansion Pressure @300 psi Exudation, psf = **95**

Minimum R-Value Requirement:

Comments:

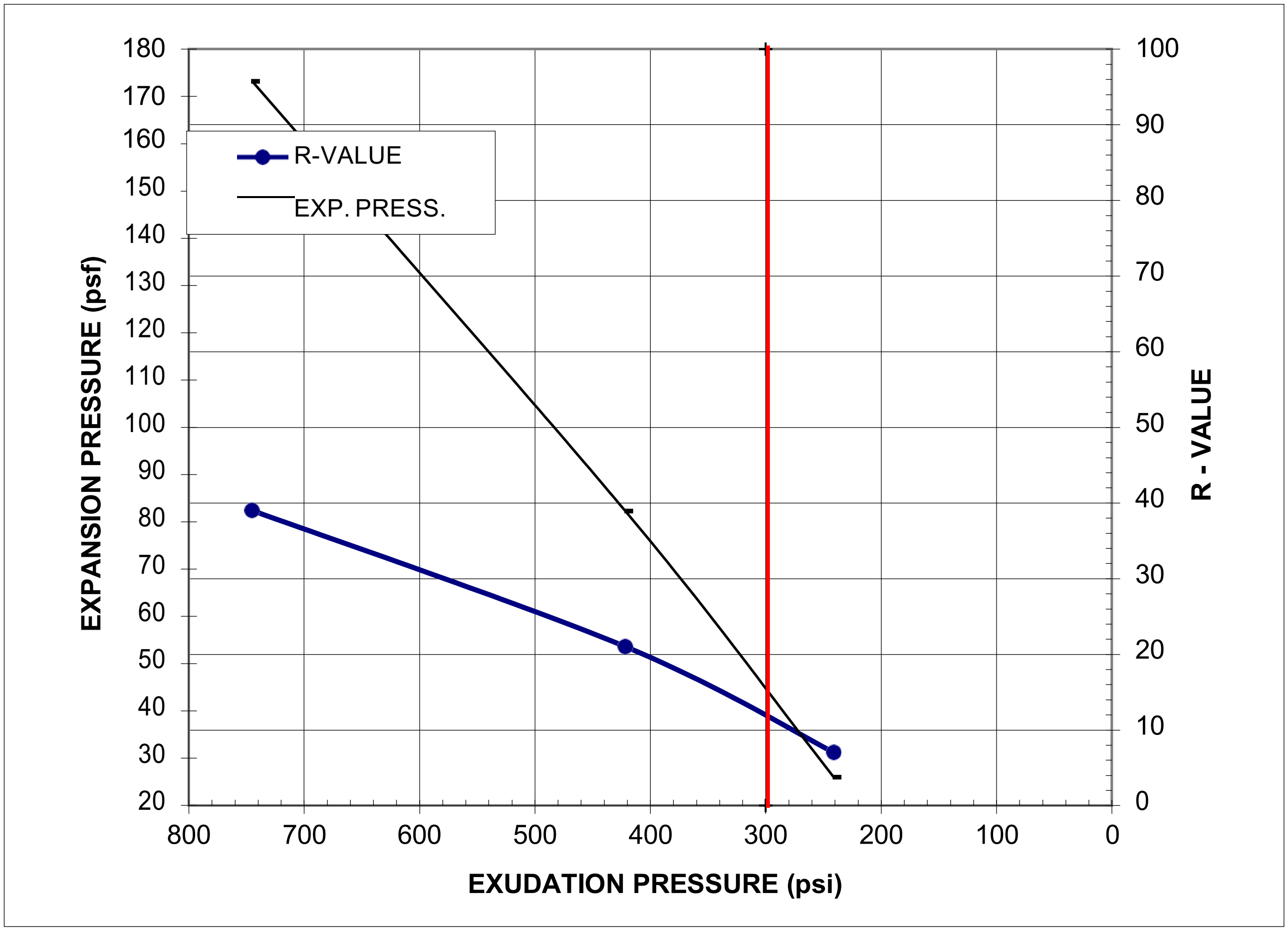
Report By: Nasir Ahmad

PLATE NO.: B-6F

R-VALUE REPORT

Parikh Consultants, Inc. **ASTM D2844 or CTM 301** (408) 452-9000

Project Name:	Main Ave and Madrone Pipelines Restoration Project			Date:	2/4/16
Client:	SCVWD			Project #:	2014-101-T03
Sample #:	B-8	Depth:	0'-5'	Lab #:	M 949
Location	Morgan Hill, CA	Onsite / Native		Sample Date:	
Material :	Sandy Clay With Gravel			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	240.94	421.6	745.1
Expansion Pressure, psf	25.98	82.27	173.2
R-Value	7	21	39
Moisture Content at Test, %	20.3	17.5	15.7
Dry Density at Test, pcf	106.2	103.0	118.5

R-Value @ 300 psi Exudation Pressure = 12

Expansion Pressure @300 psi Exudation, psf = **43**

Minimum R-Value Requirement:

Comments:

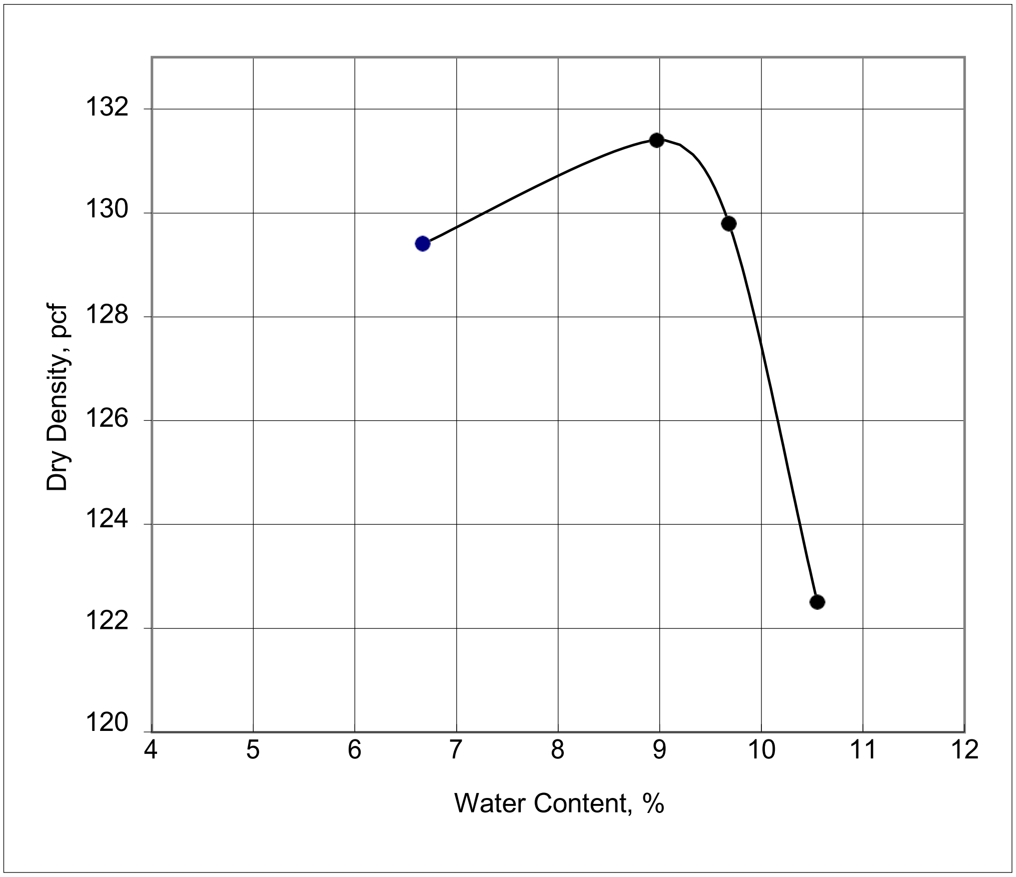
Report By: Nasir Ahmad

PLATE NO.: B-6G

LABORATORY COMPACTION REPORT

(408)-452-9000
Parikh Consultants Inc.

Project:	Main Avenue and Madrone Pipelines Restoration Project	Date:	8/28/15
Client:	Santa Clara Valley Water District	Project #:	2014-101-T03
Sample #:	B-3	Lab #:	M949
Location/Source:	Morgan Hill, California	Sample Date:	8/27/15
Material:	Sandy Lean Clay	Sampled By:	NA



ASTM Test Designation: [ ] D 698 [X] D 1557 Method: [ ] A [X] B [ ] C

100 % Saturation Curve-Estimated Specific Gravity: 2.55

Rammer Type Manual [ ] or Mechanical [X]

Sample Preparation Moist [X] Dry [ ]

Laboratory Test Results

Trial #	1	2	3	4
Water Content,%	6.7	9.0	9.7	10.6
Dry Density, pcf	129.4	131.4	129.8	122.5

MAXIMUM DRY DENSITY, pcf: **131.5**	OPTIMUM MOISTURE, %: **9.0**

Comments:

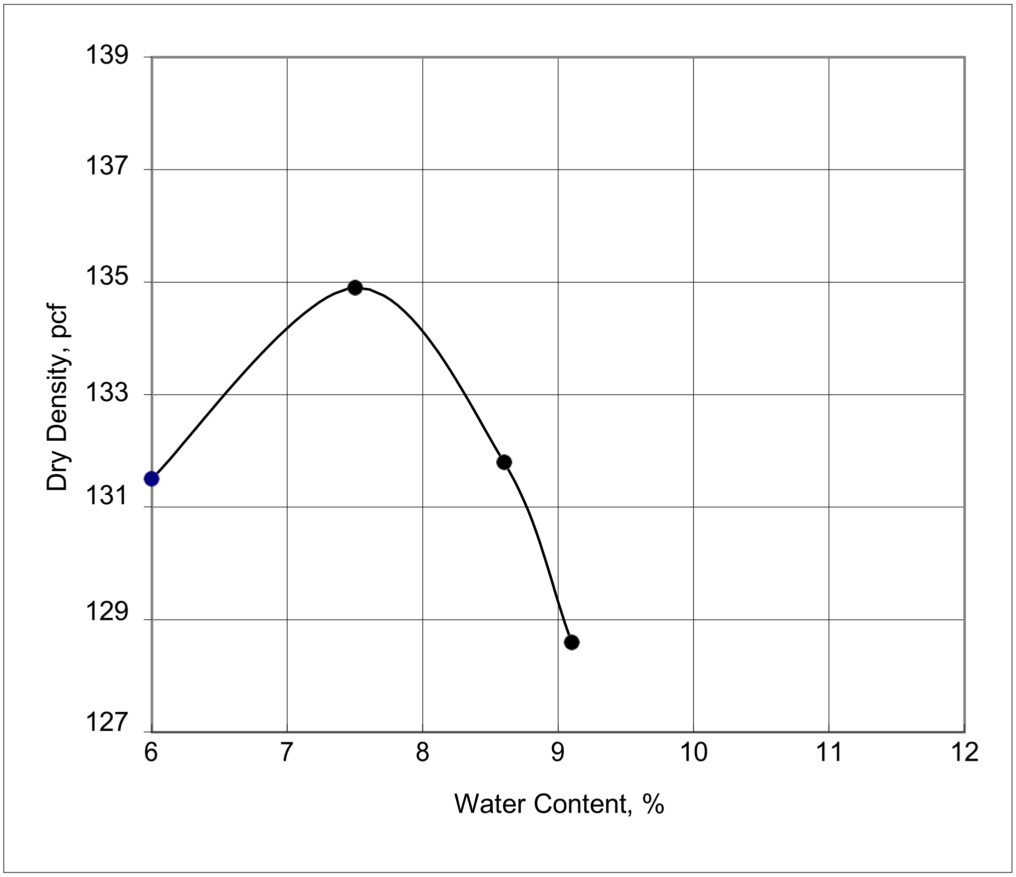
Report By Nasir Ahmad

All reports are submitted as the confidential property of our clients. Publication of statements, conclusion, or extracts is reserved pending our written approval.

LABORATORY COMPACTION REPORT

(408)-452-9000
Parikh Consultants Inc.

Project:	Main Avenue and Madrone Pipelines Restoration Project	Date:	9/14-15
Client:	Santa Clara Valley Water District	Project #:	2014-101-T03
Sample #:	B-7	Lab #:	M949
Location/Source:	Morgan Hill, California	Sample Date:	8/27/15
Material:	Silty Sand with Gravel	Sampled By:	NA



ASTM Test Designation: [ ] D 698 [X] D 1557 Method: [ ] A [X] B [ ] C

100 % Saturation Curve-Estimated Specific Gravity: 2.55

Rammer Type Manual [ ] or Mechanical [X]

Sample Preparation Moist [X] Dry [ ]

Laboratory Test Results

Trial #	1	2	3	4
Water Content,%	6.0	7.5	8.6	9.1
Dry Density, pcf	131.5	134.9	131.8	128.6

MAXIMUM DRY DENSITY, pcf: **135.0**	OPTIMUM MOISTURE, %: **7.5**

Comments:

Report By Nasir Ahmad

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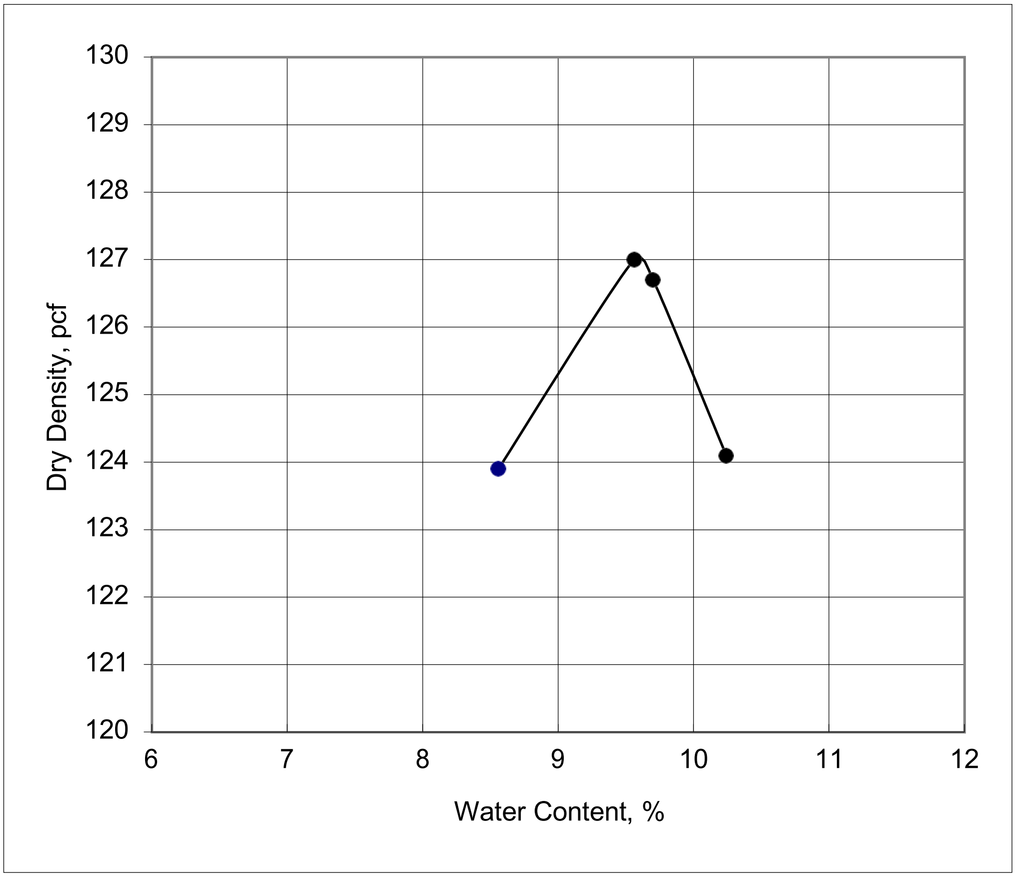
LABORATORY COMPACTION REPORT

(408)-452-9000
Parikh Consultants Inc.

Project:	Main Avenue and Madrone Pipelines Restoration	Date:	9/14-15
Client:	Santa Clara Valley Water District	Project #:	2014-101-T03
Sample #:	B-9	Lab #:	M949
Location/Source:	Morgan Hill, California	Sample Date:	8/27/15
Material:	Clayey Sand	Sampled By:	NA



ASTM Test Designation: [ ] D 698 [X] D 1557 Method: [ ] A [X] B [ ] C

100 % Saturation Curve-Estimated Specific Gravity: 2.55

Rammer Type Manual [ ] or Mechanical [X]

Sample Preparation Moist [X] Dry [ ]

Laboratory Test Results

Trial #	1	2	3	4
Water Content,%	8.6	9.6	9.7	10.2
Dry Density, pcf	123.9	127.0	126.7	124.1

MAXIMUM DRY DENSITY, pcf: **127.0**	OPTIMUM MOISTURE, %: **9.5**

Comments:

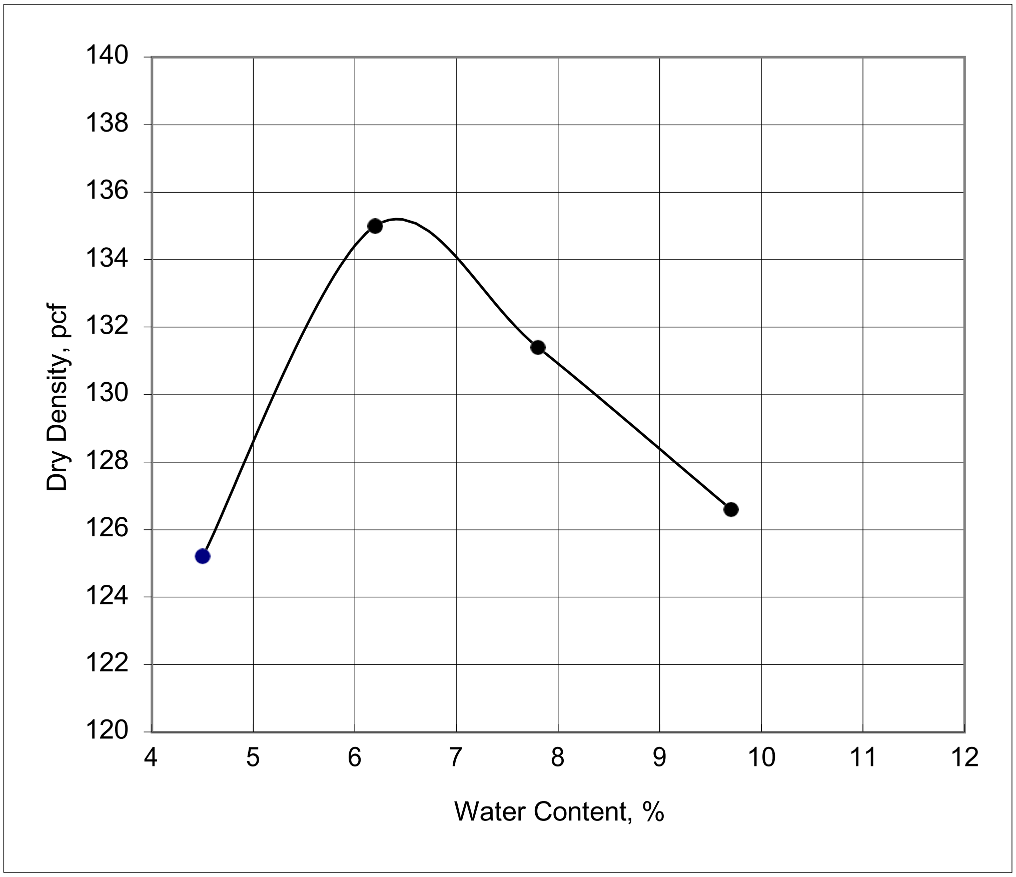
Report By Nasir Ahmad

All reports are submitted as the confidential property of our clients. Publication of statements, conclusion, or extracts is reserved pending our written approval.

LABORATORY COMPACTION REPORT

(408)-452-9000
Parikh Consultants Inc.

Project:	Main Avenue and Madrone Pipelines Restoration	Date:	9/14-15
Client:	Santa Clara Valley Water District	Project #:	2014-101-T03
Sample #:	B-12	Lab #:	M949
Location/Source:	Morgan Hill, California	Sample Date:	8/27/15
Material:	Lean Clay with Sand	Sampled By:	NA



ASTM Test Designation: [ ] D 698 [X] D 1557 Method: [ ] A [X] B [ ] C

100 % Saturation Curve-Estimated Specific Gravity: 2.55

Rammer Type Manual [ ] or Mechanical [X]

Sample Preparation Moist [X] Dry [ ]

Laboratory Test Results

Trial #	1	2	3	4
Water Content,%	4.5	6.2	7.8	9.7
Dry Density, pcf	125.2	135.0	131.4	126.6

MAXIMUM DRY DENSITY, pcf: **135.0**	OPTIMUM MOISTURE, %: **6.5**

Comments:

Report By Nasir Ahmad

All reports are submitted as the confidential property of our clients. Publication of statements, conclusion, or extracts is reserved pending our written approval.

APPENDIX C

EVALUATION OF LIQUEFACTION POTENTIAL

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME	***Main Ave and Madrone Pipelines Restoration Project***	*SOIL GROUPS*	*FAULT INFO*
PROJECT NO.	***2014-101-T03***	1. GRAVELS, SANDS AND NONPLASTIC SILTS	*SILVER CREEK FAULT*
BORING NO.	***B-01***	2. CLAYS AND PLASTIC SILTS	*a_{max} (g)=* 0.7
			FAULT M_w = 6.9

GW DEPTH (ft)= 10

BOREHOLE DIA (in)= 4
HAMMER ENERGY = 84%

CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 10 *(below OG)*

MSF = 1.24

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	$\square_v'$ (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	$\square_v$ (psf)	$\square_v'$ (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	□D (in)
0	3.5	1	2	2	55	MC	35.8	1.4	0.75	1.0	1.00	37.5	230.0	1.7	63.8										Cohesive		
3.5	8.0	2	5	2	45	MC	29.3	1.4	0.80	1.0	1.00	32.8	582.5	1.7	55.7										Plastic		
8.0	11.0	3	10	2	41	MC	26.7	1.4	0.85	1.0	1.00	31.7	1182.5	1.3	41.2										Plastic		
11.0	18.0	4	15.5	1	67	MC	43.6	1.4	0.95	1.0	1.00	57.9	1499.3	1.2	66.9	20%	75.8		1842.5	1499.3	0.97	0.5	1.0	1.0	NON-LIQ.		
18.0	21.5	5	20.5	1	81	MC	52.7	1.4	0.95	1.0	1.00	70.0	1787.3	1.1	74.1		74.1		2442.5	1787.3	0.96	0.6	1.0	1.0	NON-LIQ.		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\square_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$

   where a and b = coefficients determined from the following relationships

   for FC $\leq$ 5% a = 0, b = 1.0

   for 5% < FC < 35% a = exp(1.76-(190/FC^2)), b = (0.99+(FC$^{1.5}$/1000))

   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME	***Main Ave and Madrone Pipelines Restoration Project***	*SOIL GROUPS*		*FAULT INFO*	
PROJECT NO.	***2014-101-T03***	1. GRAVELS, SANDS AND NONPLASTIC SILTS		*SILVER CREEK FAULT*	
BORING NO.	***B-02***	2. CLAYS AND PLASTIC SILTS		*a_{max} (g)=*	0.7
				FAULT M_w =	6.9
GW DEPTH (ft)=	10	*BOREHOLE DIA (in)=*	4	*CUT(-)/FILL(+) (ft) =*	0
		HAMMER ENERGY =	84%	*DESIGN GW DEPTH (ft)=*	10 *(below OG)*
				MSF =	1.24

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	$SPT\text{-}N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	$\square_v'$ (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	$\square_v$ (psf)	$\square_v'$ (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	□D (in)
0	4.0	1	3	2	13	MC	8.5	1.4	0.75	1.0	1.00	8.9	345.0	1.7	15.1										Cohesive		
4.0	8.5	2	6	2	29	MC	18.9	1.4	0.80	1.0	1.00	21.1	700.0	1.7	35.7										Plastic		
8.5	14.0	3	11	1	42	MC	27.3	1.4	0.85	1.0	1.00	32.5	1237.6	1.3	41.3		41.3		1300.0	1237.6	0.98	0.5	1.0	1.0	NON-LIQ.		
14.0	18.0	4	16	1	50	MC	32.5	1.4	0.95	1.0	1.00	43.2	1525.6	1.1	49.5		49.5		1900.0	1525.6	0.97	0.5	1.0	1.0	NON-LIQ.		
18.0	21.5	5	21	2	38	MC	24.7	1.4	0.95	1.0	1.00	32.9	1813.6	1.1	34.5										Plastic		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\square_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$

   where a and b = coefficients determined from the following relationships

   for FC ≤ 5% a = 0, b = 1.0

   for 5% < FC < 35% $a = \exp(1.76-(190/FC^2))$, $b = (0.99+(FC^{1.5}/1000))$

   for FC ≥ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***Main Ave and Madrone Pipelines Restoration Project***
PROJECT NO. ***2014-101-T03***
BORING NO. ***B-03***

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS

FAULT INFO
SILVER CREEK FAULT
a_{max} (g)= 0.7
FAULT M_w = 6.9

GW DEPTH (ft)= 10

BOREHOLE DIA (in)= 7
HAMMER ENERGY = 84%

CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 10 *(below OG)*

MSF = 1.24

SOIL STRATA							*LIQUEFACTION RESISTANCE (CRR $_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=(CRR$_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	□v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	□v (psf)	□v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	□D (in)
0	4.0	1	3	2	58	MC	37.7	1.4	0.75	1.0	1.15	45.5	345.0	1.7	77.4										Cohesive		
4.0	9.0	2	6	2	64	MC	41.6	1.4	0.80	1.0	1.15	53.6	700.0	1.7	90.6										Plastic		
9.0	14.0	3	11	2	75	MC	48.8	1.4	0.85	1.0	1.15	66.7	1237.6	1.3	84.8										Plastic		
14.0	18.0	4	16	2	32	MC	20.8	1.4	0.95	1.0	1.15	31.8	1525.6	1.1	36.4										Plastic		
18.0	21.5	5	21	2	37	MC	24.1	1.4	0.95	1.0	1.15	36.8	1813.6	1.1	38.6										Plastic		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/□_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$

   where a and b = coefficients determined from the following relationships

   for FC $\leq$ 5% a = 0, b = 1.0

   for 5% < FC < 35% a = exp(1.76-(190/FC^2)), b = (0.99+(FC$^{1.5}$/1000))

   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME	***Main Ave and Madrone Pipelines Restoration Project***	*SOIL GROUPS*			*FAULT INFO*	
PROJECT NO.	***2014-101-T03***	1. GRAVELS, SANDS AND NONPLASTIC SILTS			*SILVER CREEK FAULT*	
BORING NO.	***B-04***	2. CLAYS AND PLASTIC SILTS			*a_{max} (g)=*	0.7
					FAULT M_w =	6.9
GW DEPTH (ft)=	10	*BOREHOLE DIA (in)=*	7	*CUT(-)/FILL(+) (ft) =* 0	*MSF =*	1.24
		HAMMER ENERGY =	84%	*DESIGN GW DEPTH (ft)=* 10 *(below OG)*		

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	$SPT\text{-}N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	□v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	□v (psf)	□v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	□D (in)
0	4.0	1	3	2	42	MC	27.3	1.4	0.75	1.0	1.15	33.0	345.0	1.7	56.0										Cohesive		
4.0	9.0	2	6	2	100	MC	65.0	1.4	0.80	1.0	1.15	83.7	700.0	1.7	141.5										Plastic		
9.0	14.0	3	11	1	36	SPT	36.0	1.4	0.85	1.2	1.15	59.1	1237.6	1.3	75.2	13%	79.8		1300.0	1237.6	0.98	0.5	1.0	1.0	NON-LIQ.		
14.0	18.0	4	16	2	85	MC	55.3	1.4	0.95	1.0	1.15	84.5	1525.6	1.1	96.8										Plastic		
18.0	21.2	5	20.5	2	100	SPT	100.0	1.4	0.95	1.2	1.15	183.5	1784.8	1.1	194.3										Plastic		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/□_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$
   where a and b = coefficients determined from the following relationships
   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% a = exp(1.76-(190/FC^2)), b = (0.99+($FC^{1.5}$/1000))
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME	***Main Ave and Madrone Pipelines Restoration Project***	*SOIL GROUPS*		*FAULT INFO*	
PROJECT NO.	***2014-101-T03***	1. GRAVELS, SANDS AND NONPLASTIC SILTS		*SILVER CREEK FAULT*	
BORING NO.	***B-05***	2. CLAYS AND PLASTIC SILTS		*a_{max} (g)=*	0.7
				FAULT M_w =	6.9

GW DEPTH (ft)=	10	*BOREHOLE DIA (in)=*	7	*CUT(-)/FILL(+) (ft) =*	0	*MSF =*	1.24
		HAMMER ENERGY =	84%	*DESIGN GW DEPTH (ft)=*	10 (below OG)		

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	σ_v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	σ_v (psf)	σ_v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	ΔD (in)
0	4.0	1	3	2	47	MC	30.6	1.4	0.75	1.0	1.15	36.9	345.0	1.7	62.7										Cohesive		
4.0	9.0	2	6	2	47	MC	30.6	1.4	0.80	1.0	1.15	39.3	700.0	1.7	66.5										Plastic		
9.0	14.0	3	11	2	30	MC	19.5	1.4	0.85	1.0	1.15	26.7	1237.6	1.3	33.9										Plastic		
14.0	18.0	4	16	2	43	MC	28.0	1.4	0.95	1.0	1.15	42.7	1525.6	1.1	48.9										Plastic		
18.0	21.5	5	21	2	81	MC	52.7	1.4	0.95	1.0	1.15	80.5	1813.6	1.1	84.6										Plastic		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\sigma_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$

   where a and b = coefficients determined from the following relationships

   for FC $\leq$ 5% a = 0, b = 1.0

   for 5% < FC < 35% $a = \exp(1.76-(190/FC^2))$, $b = (0.99+(FC^{1.5}/1000))$

   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME	***Main Ave and Madrone Pipelines Restoration Project***	*SOIL GROUPS*		*FAULT INFO*	
PROJECT NO.	***2014-101-T03***	1. GRAVELS, SANDS AND NONPLASTIC SILTS		*SILVER CREEK FAULT*	
BORING NO.	***B-06***	2. CLAYS AND PLASTIC SILTS		*a_{max} (g)=*	0.7
				FAULT M_w =	6.9
GW DEPTH (ft)=	10	*BOREHOLE DIA (in)=*	4	*CUT(-)/FILL(+) (ft) =*	0
		HAMMER ENERGY =	84%	*DESIGN GW DEPTH (ft)=*	10 *(below OG)*
				MSF =	1.24

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	σ_v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	σ_v (psf)	σ_v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	ΔD (in)
0	4.0	1	3	2	33	MC	21.5	1.4	0.75	1.0	1.00	22.5	345.0	1.7	38.3										Cohesive		
4.0	5.5	2	5	2	17	MC	11.1	1.4	0.80	1.0	1.00	12.4	580.0	1.7	21.0										Plastic		
5.5	8.5	3	5.5	1	17	MC	11.1	1.4	0.80	1.0	1.00	12.4	640.0	1.7	21.0	14%	24.1		640.0	640.0	0.99	0.5	1.0	1.0	Above GW		
8.5	9.8	4	9.5	2	100	MC	65.0	1.4	0.85	1.0	1.00	77.4	1120.0	1.3	103.4										Plastic		
9.8	20.3	5	20	1	100	SPT	100.0	1.4	0.95	1.2	1.00	159.6	1756.0	1.1	170.3		170.3		2380.0	1756.0	0.96	0.6	1.0	1.0	NON-LIQ.		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\sigma_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$

   where a and b = coefficients determined from the following relationships

   for FC $\leq$ 5% a = 0, b = 1.0

   for 5% < FC < 35% a = $\exp(1.76-(190/FC^2))$, b = $(0.99+(FC^{1.5}/1000))$

   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

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LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***Main Ave and Madrone Pipelines Restoration Project***
PROJECT NO. ***2014-101-T03***
BORING NO. ***B-07***

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS

FAULT INFO
SILVER CREEK FAULT
a_{max} (g)= 0.7
FAULT M_w = 6.9

GW DEPTH (ft)= 10

BOREHOLE DIA (in)= 7
HAMMER ENERGY = 84%

CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 10 *(below OG)*

MSF = 1.24

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer Thickness from	to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	$\square_v'$ (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	$\square_v$ (psf)	$\square_v'$ (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	□D (in)
0	4.0	1	3	1	23	MC	15.0	1.4	0.75	1.0	1.15	18.1	345.0	1.7	30.7		30.7		345.0	345.0	0.99	0.5	1.0	1.0	Above GW		
4.0	8.0	2	6	1	55	MC	35.8	1.4	0.80	1.0	1.15	46.0	700.0	1.7	77.8		77.8		700.0	700.0	0.99	0.4	1.0	1.0	Above GW		
8.0	13.0	3	11	1	38	MC	24.7	1.4	0.85	1.0	1.15	33.8	1237.6	1.3	43.0	46%	56.6		1300.0	1237.6	0.98	0.5	1.0	1.0	NON-LIQ.		
13.0	18.0	4	16	1	60	MC	39.0	1.4	0.95	1.0	1.15	59.7	1525.6	1.1	68.3		68.3		1900.0	1525.6	0.97	0.5	1.0	1.0	NON-LIQ.		
18.0	21.5	5	21	1	52	MC	33.8	1.4	0.95	1.0	1.15	51.7	1813.6	1.1	54.3	11%	56.9		2500.0	1813.6	0.95	0.6	1.0	1.0	NON-LIQ.		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\square_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$
   where a and b = coefficients determined from the following relationships
   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% a = exp(1.76-(190/FC^2)), b = (0.99+(FC$^{1.5}$/1000))
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

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LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME	***Main Ave and Madrone Pipelines Restoration Project***	*SOIL GROUPS*		*FAULT INFO*
PROJECT NO.	***2014-101-T03***	1. GRAVELS, SANDS AND NONPLASTIC SILTS		*SILVER CREEK FAULT*
BORING NO.	***B-08***	2. CLAYS AND PLASTIC SILTS		*a $_{max}$ (g)=* 0.7
				FAULT M $_w$ = 6.9

GW DEPTH (ft)=	10	*BOREHOLE DIA (in)=*	4	*CUT(-)/FILL(+) (ft) =*	0		*MSF =* 1.24
		HAMMER ENERGY =	84%	*DESIGN GW DEPTH (ft)=*	10	*(below OG)*	

SOIL STRATA							*LIQUEFACTION RESISTANCE (CRR $_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=(CRR $_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-N$_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	□v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	□v (psf)	□v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	□D (in)
0	4.0	1	3	2	19	MC	12.4	1.4	0.75	1.0	1.00	13.0	345.0	1.7	22.0										Cohesive		
4.0	8.0	2	6	2	21	MC	13.7	1.4	0.80	1.0	1.00	15.3	700.0	1.7	25.8										Plastic		
8.0	13.5	3	11	2	48	MC	31.2	1.4	0.85	1.0	1.00	37.1	1237.6	1.3	47.2										Plastic		
13.5	17.5	4	16	1	48	SPT	48.0	1.4	0.95	1.2	1.00	76.6	1525.6	1.1	87.7	14%	93.6		1900.0	1525.6	0.97	0.5	1.0	1.0	NON-LIQ.		
17.5	21.5	5	21	2	36	SPT	36.0	1.4	0.95	1.2	1.00	57.5	1813.6	1.1	60.3										Plastic		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\square_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$

   where a and b = coefficients determined from the following relationships

for FC $\leq$ 5%	a = 0,	b = 1.0
for 5% < FC < 35%	a = exp(1.76-(190/FC^2)),	b = (0.99+(FC$^{1.5}$/1000))
for FC $\geq$ 35%	a = 5.0,	b = 1.2

4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***Main Ave and Madrone Pipelines Restoration Project***
PROJECT NO. ***2014-101-T03***
BORING NO. ***B-09***

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS

FAULT INFO
SILVER CREEK FAULT
a_{max} (g)= 0.7
FAULT M_w = 6.9

GW DEPTH (ft)= 10

BOREHOLE DIA (in)= 7
HAMMER ENERGY = 84%

CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 10 *(below OG)*

MSF = 1.24

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	$SPT\text{-}N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	$\square_v'$ (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	$\square_v$ (psf)	$\square_v'$ (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	□D (in)
0	4.0	1	3	1	72	MC	46.8	1.4	0.75	1.0	1.15	56.5	345.0	1.7	96.1	32%	117.3		345.0	345.0	0.99	0.5	1.0	1.0	Above GW		
4.0	6.0	2	5	2	36	SPT	36.0	1.4	0.80	1.2	1.15	55.6	580.0	1.7	94.6										Plastic		
6.0	9.0	3	8	1	30	MC	19.5	1.4	0.85	1.0	1.15	26.7	940.0	1.5	38.9	18%	44.7		940.0	940.0	0.98	0.4	1.0	1.0	Above GW		
9.0	14.0	4	10	2	100	SPT	100.0	1.4	0.85	1.2	1.15	164.2	1180.0	1.3	213.8										Plastic		
14.0	18.0	5	16	2	56	SPT	56.0	1.4	0.95	1.2	1.15	102.8	1525.6	1.1	117.7										Plastic		
18.0	20.8	6	20.5	2	100	SPT	100.0	1.4	0.95	1.2	1.15	183.5	1784.8	1.1	194.3										Plastic		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\square_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$

   where a and b = coefficients determined from the following relationships

   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% a = exp(1.76-(190/FC^2)), b = (0.99+(FC$^{1.5}$/1000))
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

4/14/2016

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME	***Main Ave and Madrone Pipelines Restoration Project***	*SOIL GROUPS*			*FAULT INFO*	
PROJECT NO.	***2014-101-T03***	1. GRAVELS, SANDS AND NONPLASTIC SILTS			*SILVER CREEK FAULT*	
BORING NO.	***B-10***	2. CLAYS AND PLASTIC SILTS			*a_{max} (g)=*	0.7
					FAULT M_w =	6.9
GW DEPTH (ft)=	10	*BOREHOLE DIA (in)=*	7	*CUT(-)/FILL(+) (ft) =* 0	*MSF =*	1.24
		HAMMER ENERGY =	84%	*DESIGN GW DEPTH (ft)=* 10 *(below OG)*		

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	□$_v$' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	□$_v$ (psf)	□$_v$' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	□D (in)
0	4.0	1	3	2	12	MC	7.8	1.4	0.75	1.0	1.15	9.4	345.0	1.7	16.0										Cohesive		
4.0	8.5	2	6	2	17	MC	11.1	1.4	0.80	1.0	1.15	14.2	700.0	1.7	24.1										Plastic		
8.5	13.5	3	11	2	33	MC	21.5	1.4	0.85	1.0	1.15	29.4	1237.6	1.3	37.3										Plastic		
13.5	18.0	4	16	1	46	MC	29.9	1.4	0.95	1.0	1.15	45.7	1525.6	1.1	52.4	14%	56.8		1900.0	1525.6	0.97	0.5	1.0	1.0	NON-LIQ.		
18.0	21.5	5	21	1	62	SPT	62.0	1.4	0.95	1.2	1.15	113.8	1813.6	1.1	119.5		119.5		2500.0	1813.6	0.95	0.6	1.0	1.0	NON-LIQ.		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, C_N = (1/□$_v$')$^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$

   where a and b = coefficients determined from the following relationships

for FC $\leq$ 5%	a = 0,	b = 1.0
for 5% < FC < 35%	a = exp(1.76-(190/FC^2)),	b = (0.99+(FC$^{1.5}$/1000))
for FC $\geq$ 35%	a = 5.0,	b = 1.2

4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***Main Ave and Madrone Pipelines Restoration Project***
PROJECT NO. ***2014-101-T03***
BORING NO. ***B-11***

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS

FAULT INFO
SILVER CREEK FAULT
a_{max} (g)= 0.7
FAULT M_w = 6.9

GW DEPTH (ft)= 10

BOREHOLE DIA (in)= 7
HAMMER ENERGY = 84%

CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 10 *(below OG)*

MSF = 1.24

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	□v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	□v (psf)	□v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	□D (in)
0	4.0	1	3	2	15	MC	9.8	1.4	0.75	1.0	1.15	11.8	345.0	1.7	20.0										Cohesive		
4.0	9.0	2	6	2	30	MC	19.5	1.4	0.80	1.0	1.15	25.1	700.0	1.7	42.5										Plastic		
9.0	14.0	3	11	2	70	MC	45.5	1.4	0.85	1.0	1.15	62.3	1237.6	1.3	79.2										Plastic		
14.0	18.0	4	15.5	2	100	MC	65.0	1.4	0.95	1.0	1.15	99.4	1496.8	1.2	114.9										Plastic		
18.0	21.5	5	21	2	85	SPT	85.0	1.4	0.95	1.2	1.15	156.0	1813.6	1.1	163.8										Plastic		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, C_N = (1/□v')$^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs}$ = a + b $(N_1)_{60}$
   where a and b = coefficients determined from the following relationships
   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% a = exp(1.76-(190/FC^2)), b = (0.99+(FC$^{1.5}$/1000))
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME	***Main Ave and Madrone Pipelines Restoration Project***	*SOIL GROUPS*			*FAULT INFO*	
PROJECT NO.	***2014-101-T03***	1. GRAVELS, SANDS AND NONPLASTIC SILTS			*SILVER CREEK FAULT*	
BORING NO.	***B-12***	2. CLAYS AND PLASTIC SILTS			*a $_{max}$ (g)=*	0.7
					FAULT M $_w$ =	6.9
GW DEPTH (ft)=	10	*BOREHOLE DIA (in)=*	7	*CUT(-)/FILL(+) (ft) =* 0	*MSF =*	1.24
		HAMMER ENERGY =	84%	*DESIGN GW DEPTH (ft)=* 10 *(below OG)*		

SOIL STRATA							*LIQUEFACTION RESISTANCE (CRR $_{7.5}$)*													*CYCLIC STRESS RATIO (CSR)*				*F.S.=(CRR $_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-N$_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	□$_v$' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60, CS}$	$CRR_{7.5}$	□$_v$ (psf)	□$_v$' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	□D (in)
0	4.0	1	3	2	15	MC	9.8	1.4	0.75	1.0	1.15	11.8	345.0	1.7	20.0										Cohesive		
4.0	9.0	2	6	1	30	MC	19.5	1.4	0.80	1.0	1.15	25.1	700.0	1.7	42.5	22%	50.3		700.0	700.0	0.99	0.4	1.0	1.0	Above GW		
9.0	14.0	3	11	1	70	MC	45.5	1.4	0.85	1.0	1.15	62.3	1237.6	1.3	79.2		79.2		1300.0	1237.6	0.98	0.5	1.0	1.0	NON-LIQ.		
14.0	18.0	4	15.5	1	100	MC	65.0	1.4	0.95	1.0	1.15	99.4	1496.8	1.2	114.9		114.9		1840.0	1496.8	0.97	0.5	1.0	1.0	NON-LIQ.		
18.0	21.5	5	21	1	85	SPT	85.0	1.4	0.95	1.2	1.15	156.0	1813.6	1.1	163.8		163.8		2500.0	1813.6	0.95	0.6	1.0	1.0	NON-LIQ.		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, C_N = (1/□$_v$')$^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$

   where a and b = coefficients determined from the following relationships

   for FC $\leq$ 5%   a = 0,   b = 1.0

   for 5% < FC < 35%   a = exp(1.76-(190/FC^2)), b = (0.99+(FC$^{1.5}$/1000))

   for FC $\geq$ 35%   a = 5.0,   b = 1.2
4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

4/14/2016

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME	***Main Ave and Madrone Pipelines Restoration Project***	*SOIL GROUPS*		*FAULT INFO*	
PROJECT NO.	***2014-101-T03***	1. GRAVELS, SANDS AND NONPLASTIC SILTS		*SILVER CREEK FAULT*	
BORING NO.	***B-13***	2. CLAYS AND PLASTIC SILTS		*a_{max} (g)=*	0.7
				FAULT M_w =	6.9
GW DEPTH (ft)=	10	*BOREHOLE DIA (in)=*	7	*CUT(-)/FILL(+) (ft) =*	0
		HAMMER ENERGY =	84%	*DESIGN GW DEPTH (ft)=*	10 *(below OG)*
				MSF =	1.24

SOIL STRATA							*LIQUEFACTION RESISTANCE (CRR $_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=(CRR $_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	□v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	□v (psf)	□v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	□D (in)
0	4.0	1	3	2	34	MC	22.1	1.4	0.75	1.0	1.15	26.7	345.0	1.7	45.4										Cohesive		
4.0	9.0	2	6	2	100	MC	65.0	1.4	0.80	1.0	1.15	83.7	700.0	1.7	141.5										Plastic		
9.0	14.0	3	11	2	48	MC	31.2	1.4	0.85	1.0	1.15	42.7	1237.6	1.3	54.3										Plastic		
14.0	18.0	4	15.5	2	100	MC	65.0	1.4	0.95	1.0	1.15	99.4	1496.8	1.2	114.9										Plastic		
18.0	21.5	5	21	1	34	SPT	34.0	1.4	0.95	1.2	1.15	62.4	1813.6	1.1	65.5	18%	73.1		2500.0	1813.6	0.95	0.6	1.0	1.0	NON-LIQ.		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/□_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\,(N_1)_{60}$

   where a and b = coefficients determined from the following relationships

   for FC $\leq$ 5% a = 0, b = 1.0

   for 5% < FC < 35% a = exp(1.76-(190/FC^2)), b = (0.99+(FC$^{1.5}$/1000))

   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***Main Ave and Madrone Pipelines Restoration Project***
PROJECT NO. ***2014-101-T03***
BORING NO. ***B-14***

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS

FAULT INFO
SILVER CREEK FAULT
a_{max} (g)= 0.7
FAULT M_w = 6.9

GW DEPTH (ft)= 10

BOREHOLE DIA (in)= 7
HAMMER ENERGY = 84%

CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 10 *(below OG)*

MSF = 1.24

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer from	Thickness to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	SPT-$N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	□$_v$' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	□$_v$ (psf)	□$_v$' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	□D (in)
0	4.0	1	3	2	41	MC	26.7	1.4	0.75	1.0	1.15	32.2	345.0	1.7	54.7										Cohesive		
4.0	8.0	2	6	2	51	MC	33.2	1.4	0.80	1.0	1.15	42.7	700.0	1.7	72.2										Plastic		
8.0	13.0	3	11	1	72	MC	46.8	1.4	0.85	1.0	1.15	64.0	1237.6	1.3	81.4	16%	88.6		1300.0	1237.6	0.98	0.5	1.0	1.0	NON-LIQ.		
13.0	17.5	4	16	1	32	SPT	32.0	1.4	0.95	1.2	1.15	58.7	1525.6	1.1	67.2		67.2		1900.0	1525.6	0.97	0.5	1.0	1.0	NON-LIQ.		
17.5	21.5	5	21	1	41	SPT	41.0	1.4	0.95	1.2	1.15	75.3	1813.6	1.1	79.0		79.0		2500.0	1813.6	0.95	0.6	1.0	1.0	NON-LIQ.		

Notes:

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, C_N = (1/□$_v$')$^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs}$ = a + b $(N_1)_{60}$

   where a and b = coefficients determined from the following relationships

   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% a = exp(1.76-(190/FC^2)), b = (0.99+(FC$^{1.5}$/1000))
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 25, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:
Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

THRUST BLOCK ANALYSIS

PROJECT NO. <2 0I -III 1 -ros

PROJECT NAME I"I a',I1 A 'II.t, Md M.JfA/'I(. f L 111v.

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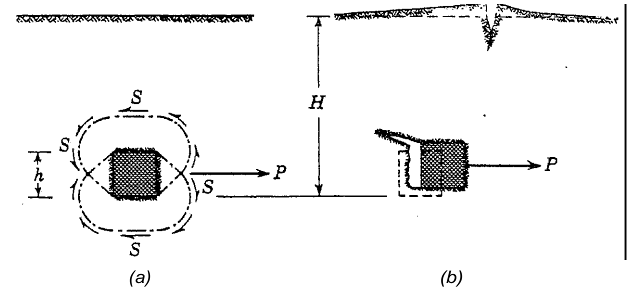
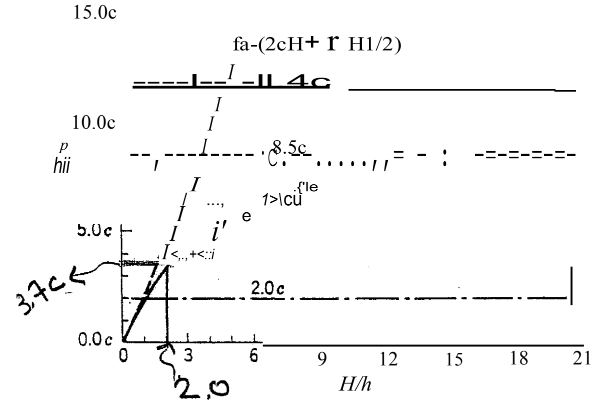
It was assumed that the clay cannot change its volume and must therefore flow around the block, offering to such motion a shearing resistance $s = c$ along the slip surfaces shown by dash-dotted lines. The total



Fm. 5-20. Failure surfaces in clay around anchor blocks below soil surface. *a,* theoretical considerations by Hansen (15); *b,* experimental results by Mackenzie (23).



Fm. 5-30. Resistance of plastic clay to anchor blocks. Experimental and theoretical results. *[After Mackenzie* (23).]

resistance P per unit of anchor-block width will be

$$P = ll.4ch \qquad (5\text{-}14)$$

Mackenzie (23) performed small-scale-model tests with two plastic clays. In these tests the *-yh* component of Eq. (5-13) was quite small as compared with the 2c value. Possibly for that reason the deformations of the clay around the anchor block were as shown in Fig. 5-29b, and

TRENCH BOTTOM HEAVE STABILITY ANALYSIS

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VERIFIED BY ________ DATE ____
BACKCHECKED BY ________ DATE ____

SUBJECT Trench Bottom Stability

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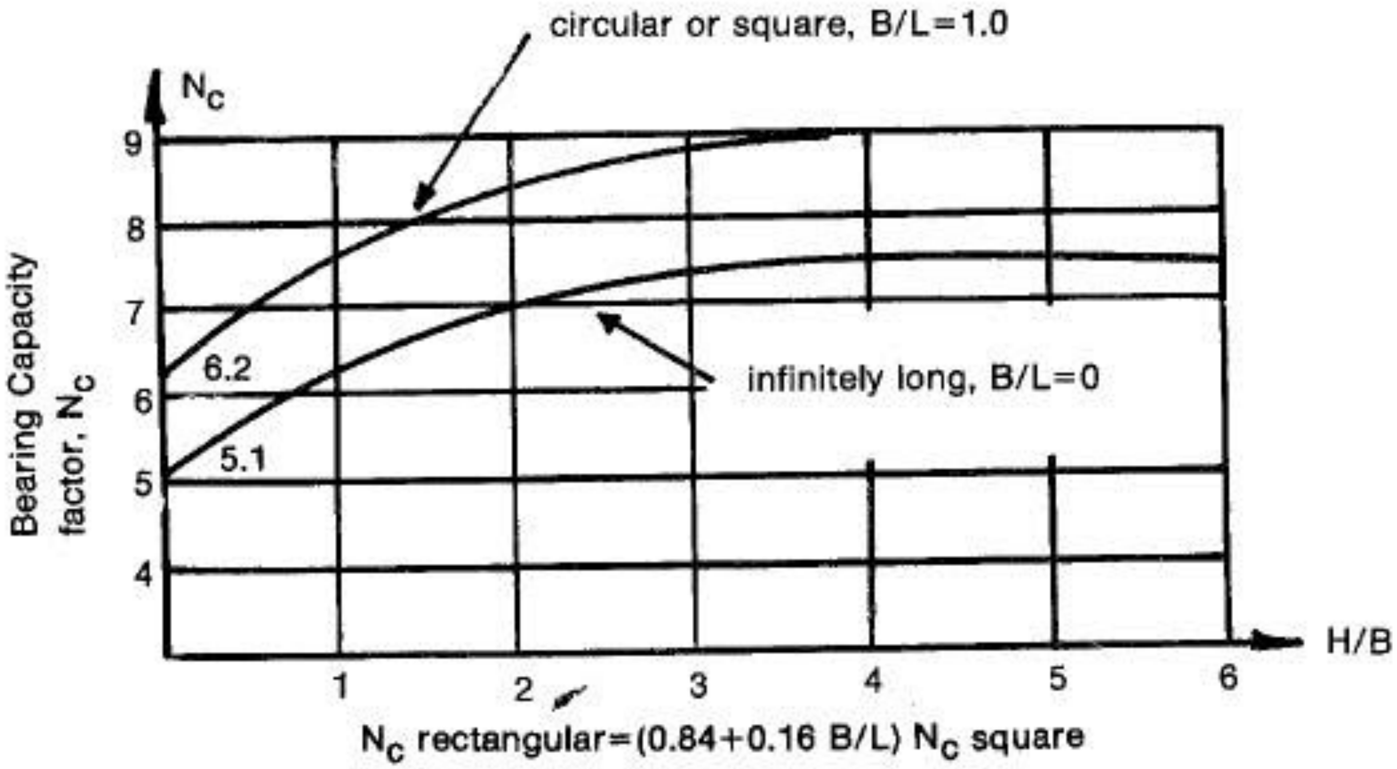


N_c rectangular=(0.84+0.16 B/L) N_c square

Figure 56. Diagram for the Determination of Bearing Pre

STRUCTURAL PAVEMENT DESIGN

PAVEMENT DESIGN

PER HIGHWAY DESIGN MANUAL, CHAP. 600

PROJECT NAME: *Main Avenue and Madrone Pipelines Restoration Project*
PROJECT NO.: *2014-101-TO3*

Design Case: AC over AB

Design TI= ***8*** input
R_{BS}= ***10*** input
R_{AB}= 78

$GE_{AC+AB} = 0.0032*TI*(100-R_{BS}) =$ 2.30

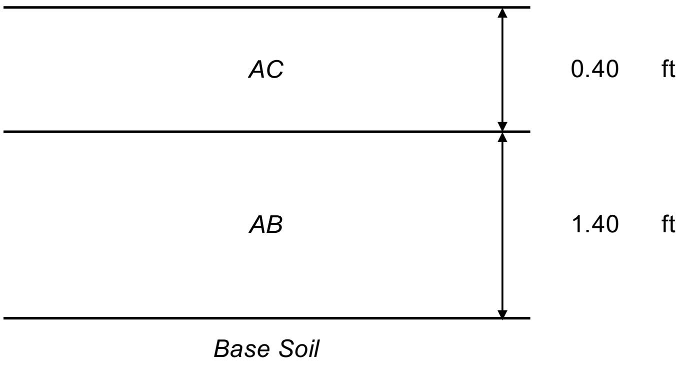
$GE_{AC} = 0.0032*TI*(100-R_{AB}) =$ 0.56
=> $GE'_{AC} =$ 0.76 (add 0.2 ft safety factor)
AC Thickness = 0.38 ft

=> AC Thickness = **0.40** ft (round up to the nearest 0.05 ft)
$G_{f, AC} =$ 2.00
$GE_{AC} =$ 0.80

$GE_{AB} = GE_{AC+AB} - GE_{AC} =$ 1.50
AB thickness= 1.37 ft

=> AB Thickness= **1.40** ft (round up to the nearest 0.05 ft)
GE_{AB}= 1.54 $G_{f, AB}$=1.1

Design Section:



CITY Of MORGAN HIU

1.100 GENERAL

All streets shall be designed in accordance with accepted engineering principles and shall conform to these Design Standards.

1.200 STREET RIGHTS OF WAY

Right-of-way widths and typical sections for various classes of streets, including private roads, shall conform to the latest edition of the City of Morgan Hill Standard Details For Construction.

1.300 ALLEY RIGHTS OF WAY

The minimum width of an alley shall be 20 feet with a 20-foot paved roadway.

1.400 STRUCTURAL SECTION

1.401 Subgrade Relative compaction for at least the top 8 inches of subgrade shall be 95%. In areas of fill, a minimum of24 inches from finished grade shall be compacted to 95% relative compaction.

1.402 Pavement Pavement shall be designed in accordance with the procedures contained in the State of California Department of Transportation Highway Design Manual.

The Traffic Index shall be established by the Public Works Department.

A certified soils laboratory shall conduct the necessary soils report and shall recommend the pavement section and address the relative expansiveness of the soil.

The minimum structural section shall be 4-inches of asphalt concrete over 8-inches of aggregate base or an alternative full-depth asphalt concrete section of7-inches, if approved by City.

1.500 HORIZONTAL ALIGNMENT

1.501 Intersection Angle Streets shall intersect at right angles. Curved streets shall have at least 50 feet of centerline tangent from the projected curb line of the intersecting street.

1.502 Opposing Streets All streets entering upon opposite sides of any given street shall have their centerline directly opposite each other or separated by at least 150 feet.